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Of The
WESTERN CANADA WATER
And
WASTEWATER ASSOCIATION

WINNIPEG, MANITOBA
September 25, 26, 27, 1991

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WESTERN CANADA POLLUTION CONTROL ASSOC. W.P.C.F.

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TABLE OF CONTENTS

PROCEEDINGS

OPERATIONS PAPERS

MANAGING A NORTHERN WATER SYSTEM FOR FREEZING PROTECTION - TOWN OF THE PAS MANITOBA 1
C.C. Macey, UMA Engineering
N. Fulford, Town of The Pas

THE WATER AUDIT AND THE ECONOMICS OF LEAKAGE CONTROL 7
D. Keeling, Heath Consultants Ltd

POLYMORPHIC RESIN COATINGS FOR USE IN WASTEWATER HANDLING FACILITIES 15
D. Whitmore, Vector Construction Ltd

SULPHUR DIOXIDE DECHLORINATION OF WASTEWATER 21
S. Bernhardt, ICI Canada Inc

LARGE DIAMETER PIPE CLEANING - PROBLEMS & SOLUTIONS 29
D. Derlein, Live Line Ltd

BATCH FLUORIDATION OF WATER STORAGE RESERVOIRS - A MODIFIED PROCESS 31
C. Marianayagam, Stan-Mar Engineering
R. Kent, Community Works, Management and Training, NWT

THE BENEFITS OF SELF PRIMING SEWAGE PUMPS IN LIFT STATIONS 39
F. Rutledge, Gorman-Rupp of Canada

HDPE REHABILITATION OF CURVED SEWER LINES WITHOUT FLOW INTERRUPTION 47
L. Sparks, Plexco/Spirolite
H. Smith, Plexco/Spirolite
A. Heinisch, Plexco/Spriolite

PRODUCTIVITY MEASUREMENT: A CASE STUDY OF THE CITY OF WINNIPEG'S WATERWORKS MAINTENANCE CREWS 53
S. Siu, City of Winnipeg

WATERMAIN LEAKAGE PREVENTION AND CONTINUING PROBLEMS AFTER REPLACEMENT WITH PLASTIC 59
J. Yule, Colony-Viking Services Ltd

TRENCHLESS REPLACEMENTS - CITY PERSPECTIVE 71
T.L. Prince, City of Calgary

AUTOMATION IN THE WASTEWATER INDUSTRY WITH COMPUTER CONTROL 85
D. Smordin, Fischer and Porter (Canada) Ltd

OPERATION OF THE BANFF TOWNSITE SEWAGE TREATMENT FACILITY 87
D.J. Taniguchi, Reid Crowther & Partners Ltd.

ADVANCES IN WASTEWATER SAMPLING 91
C. Baker, American Sigma, Inc

TECHNICAL PAPERS

A WATER SUPPLY STRATEGY FOR THE PEMBINA TRIANGLE 99
G.H. Mackay, Winnipeg,MB

LAND QUALITY AS AFFECTED BY THE WASTEWATER TREATMENT LAGOON AND DISCHARGE ROUTE AT ST. CLAUDE, MANITOBA 117
W. Michalyna, Canada-Manitoba Soil Survey, Winnipeg

DESIGN AND STARTUP OF THE PENTICTON WASTEWATER TREATMENT PLANT 123
B. Rabinowitz, Stanley Associates Engineering Ltd
G. McGeachie, Stanley Associates Engineering Ltd
B. Udala, City of Penticton

DESIGN AND START-UP OF THE MEDICINE HAT WASTEWATER TREATMENT PLANT 135
R. Johnson, Stanley Associates Engineering Ltd

CAUSES AND CONTROL OF FILAMENTOUS BACTERIA IN ACTIVATED SLUDGE: MICROSCOPIC OBSERVATION NUMBER PROVIDES EARLY DETECTION OF BULKING EPISODES 149
D.M. McCartney, Poetker MacLaren Lavalin Inc

DISINFECTION OF WASTEWATER AND ULTRAVIOLET LIGHT 157
Dr. G. Elliott, Fischer & Porter Limited
G. Palmateer, Ontario Ministry of the Environment

TYLEHURST SEWER RELIEF AND POLLUTION ABATEMENT STUDIES - CITY OF WINNIPEG, MANITOBA 169
C.C. Macey, UMA Engineering Ltd
A. Nagy, UMA Engineering Ltd

AN OVERVIEW OF THE APPLICATIONS OF OZONE 177
R. Larocque, Hankin Atlas Ozone Systems Limited

RECOVERY OF GIARDIA CYSTS USING HOLLOW FIBRE FILTRATION 185
J. Mak, City of Calgary
C.M Yentsch, Bigelow Laboratory for Ocean Sciences
E.E. Hargesheimer, City of Calgary
C.M. Lewis, City of Calgary

REMOVAL OF THM PRECURSORS FROM AN AGRICULTURAL SURFACE WATER SUPPLY 191
D. Hidajat, University of Manitoba
D. Woytowich, University of Manitoba
A. Sparling, University of Manitoba

STANDARDS AND APPROVALS FOR MUNICIPAL WATER SUPPLY SYSTEMS IN ALBERTA 203
K. Chinniah, Alberta Environment
B. Aidun, Alberta Environment
D. Spink, Alberta Environment

REMEDIATION OF GASOLINE CONTAMINATED GROUND WATER 221
E. Wolowich, Wardrop Engineering Inc.
B. Horning, Wardrop Engineering Inc.

PROPER APPLICATION OF PROGRESSING CAVITY PUMPS 233
D. Meffen, Robbins & Myers Canada Ltd

PAPERS NOT INCLUDED IN THIS PUBLICATION

The following papers were not available at the time of printing. Copies will be available from the author at the time the paper is being presented.

TWIN RIVER CROSSING INSPECTION
Siri Fernando, City of Edmonton

OPTIMIZATION OF UPFLOW TYPE SOLIDS CONTACT CLARIFIERS AT THE E.L. SMITH WATER TREATMENT PLANT, EDMONTON, ALBERTA
D. Kellendonk, City of Edmonton

FATE OF PRIORITY ORGANICS IN WASTEWATER TREATMENT PROCESSES
J.A. Oleszkiewicz, University of Manitoba
K. Themel, University of Manitoba
D.E. Sullivan, Searle Inc,

TURBIDITY AND COLOUR REMOVAL USING A HIGH-RATE CONTACT ADSORPTION CLARIFICATION - MIXED MEDIA FILTRATION PROCESS ON HIGH COLOUR CANADIAN WATERS
D. Garon, CPC Microfloc

HEAT DRYING SLUDGE
J. Medhurst, Envirex Inc

MANAGING A NORTHERN WATER SYSTEM
FOR FREEZING PROTECTION
TOWN OF THE PAS, MANITOBA

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Town Of The Pas

INTRODUCTION

The Town of The Pas is a community in Northern Manitoba. Beyond the 53rd parallel, the community is approximately 650 km (by road) north of The City of Winnipeg. The Pas's history literally pre-dates present day development in Western Canada. The area was inhabited by The Cree long before the first white man, Henry Kelsey, passed through in 1691 on his way to the prairies. The first permanent non-Indian settlement at the site was built between 1741 and 1750 as The Pas became an important link in the Northwestern Canadian Fur Trade industry. The present day community has a diversified economy acting as a centre for agricultural activities to the west and the south and the logging/pulp and paper industry to the north.

The Town was legally incorporated in 1912 and has a present day population of approximately 7300. The water supply and distribution system was originally constructed in 1913. The original installation included approximately 3.6 km (12,000 feet) of steel watermain in diameters ranging from 150 mm to 300 mm and was one of the earliest pressurized distribution systems in operation in the Province of Manitoba.

The intake well structure to the water treatment plant and some of the steel distribution mains from the original 1913 construction are still in operation. The existing water treatment plant has a maximum capacity of 4.5 Ml/day (2.24 million Imperial gallons/day) and an adjoining reservoir with 1.4 Ml (500,000 Imperial gallons) of storage capacity.

Water is heated in the winter months at the treatment plant prior to distribution. The only method of freezing protection in the distribution system is by bleeding water at services and on dead end watermains. Numerous private water services freeze annually and used to be thawed (or attempted to be thawed) by means of a direct current (DC) welder.

By the early 1970's a number of problems were recognized with respect to operation of the system. The number of frozen services annually had risen to an unacceptable level. The extensive use of the electric welder for thawing purposes was becoming a serious safety concern. Water consumption during the winter months was so high due to the widespread practice of bleeding that it approached maximum plant capacity. With water consumption so high it also appeared that the existing boiler was of insufficient capacity to raise water temperature to a desirable level prior to distribution.

In 1981 the Town applied to the Public Utilities Board for a rate increase as the water utility was running at a significant and annually increasing deficit. During that review process it became evident that steps must be taken to gain a tighter control on the system operation. Even though all water services are metered in The Pas, the 1980 data set showed that 64% of all water produced at the plant was unaccounted for.

BACKGROUND

The water distribution system in The Pas is a conventional uninsulated system. The existing system is now comprised primarily of cast iron mains. PVC watermains have been installed in new developments (approximately since 1983), however, they form a relatively small portion of the overall inventory as does the remaining steel pipe from the 1913 installation. The vast majority of building service connections are copper pipe (some lead services remain in older areas) and are equipped with a copper thawing cable connected from the corporation stop at the main and terminating at the curb box on property line.

The practice of bleeding in conjunction with heat applied at the source has long been the sole method of freezing protection in The Town and was recognized as the most cost effective form of system operation well into the 1960's. Commencing in the late 1960's and throughout the 1970's the Town undertook a large number of street paving projects upgrading existing gravel roads to an asphalt surface complete with concrete cub and gutter sections.

Existing gravel surfaces were typically over thicknesses of 0.30 m to 0.60 m and greater of muskeg. The paving program resulted in the removal of all existing muskeg within the street right-of-way as well as final pavement elevations lower than the previous surface elevations (to permit adequate longitudinal drainage). The muskeg had previously acted somewhat as an insulator retarding the depth of frost penetration annually while the new pavement grades lowered the height of cover over existing water services. As a result the incidence of frozen water services and the magnitude of water consumption during even average winters increased markedly during the 1970's.

RATIONALIZATION OF PROBLEMS

Attempting to rationalize the operational problems that existed in the early 1980's included consideration of the following:

1. The distribution system by 1980 was quite large with the incidence of frozen services and bleeding so widespread that massive retrofitting programs to minimize heat loss in the system would likely be prohibitively expensive. However, if an increase in plant capacity was required solely to meet bleeder demand the viability of bleeding as a means of freezing protection would have to be seriously reviewed.

2. Increasing water temperature at the source would be expensive due to the requirement for additional boiler capacity. More heat also translated to more heat loss and it was believed there would be at best only minimal reduction in the incidence of frozen services.

3. It was noted that bleeder use was indiscriminate. There was no formal program for bleeder operation. The number and locations of bleeders in use annually was not known. The rate of bleeding was not monitored and the length of operation of each bleeder was based on experience but would be heavily biased by the outside air temperature (ie. they would likely be turned on annually long before the depth of frost penetration warranted). There was also no formal program to turn bleeders off for the summer.

4. The procedure for boiler operation was similar to bleeder operation. The boiler was turned on in mid-November annually as air temperatures dropped and turned off in late April or early May based on experience. There was no hard data to suggest when and when not to operate the boiler.

5. It was felt that no matter what improvements were carried out to the distribution system in the short term there would still be a requirement to thaw frozen water lines for a number of years. The continued use of the DC welder for this purpose was considered to be an unacceptable risk.

   While common in use at that time and still in use today a conventional DC welder is not recommended for thawing municipal services. The high current generated creates a tremendous potential for stray currents and the development of current paths that are considerably different than the intended current path. Numerous cases of unintentional damage have been documented ranging from extensive electrical panel damage up to burning houses to the ground at locations well away from the thawing activities.

6. The use of the DC welder in The Pas was taking on an increasing risk as thawing cables on numerous services could not be located. Attempts to thaw services with current paths along the iron mains has considerably more potential to cause inadvertent damage in residences well away from the main or house service intended to be thawed.

Based on these considerations, significant improvements in system operation had to be achieved in three areas, otherwise major system upgrading would be required at costs well beyond the tax base of the community. These three areas included:

1. Reducing water consumption through bleeder control and a more rigorous approach in monitoring consumption to minimize unaccounted for water.

2. Optimization of the boiler operation to ensure that water is not heated during time periods where the depth of frost penetration is well above the depth of services.

3. Development of a reliable and safe program for the thawing of frozen building services.

SOLUTIONS AND RESULTS

1. Bleeding and Water Consumption

The first obvious step in a bleeder control program was to locate all bleeders and obtain the history of why they were installed and how they are operated. In excess of 600 bleeders were located most of which were in operation annually. Typical bleeder installation history and operation was as follows:

- most residential bleeders were originally installed and operated annually in response to a frozen service.
- a number of residential bleeders were installed and operated annually as a result of the service to a residence on their street freezing other than their own.
- where bleeders were consciously operated annually they were turned on in reaction to air temperatures long before the depth of frost penetration warranted.
- distribution main bleeders existed on mains with a freezing history, on dead-ended mains regardless of freezing history, on hydrant leads where a freezing history existed, and in some areas to prevent sewers from freezing.

To provide guidance as to when bleeders should and should not be turned on, the Town forces installed a number of frost probes at various locations in the community. These probes would monitor the depth of frost penetration as the winter progressed each year. The locations were selected based on past freezing records showing the areas with the greatest potential for freezing.

The keeping of detailed maintenance records on watermain breaks in The Pas commenced in the mid-1970's. Data in these records noting the location of watermain breaks, the depth of cover on the main, and the depth of frost penetration (if applicable) proved to be invaluable once compiled and used in conjunction with the frost probe data.

The use of numerous bleeders in the system was discontinued. While some bleeder operation is initiated annually in response to a frozen service the vast majority of bleeders are turned on prior to freezing in response to the known depth of frost penetration and the data base that has been developed on previous freezing history and depth of services. This has greatly reduced the number of services that require thawing annually. The number of bleeders that operate annually varies with the severity of the winter, however, approximately 200 will be in operation in an average year - which is a considerable reduction from the over 600 bleeders in operation prior to the commencement of the program.

The flow rate of all bleeders is monitored. Based on experience in water systems in the Yukon Territories a flow rate of approximately 1.2 l/min (0.25 Igpm) was selected as a bleeding rate for bleeders tapped into a conventional 20 mm diameter house service. While experience has shown some services may require a greater bleeding rate or can operate satisfactorily with a lower bleeding rate, it has proved to be a good rule of thumb as a starting point to determine the optimum bleeding rate.

The results of the bleeder control program have been dramatic. The first frost probes were installed in late 1981. Compilation of data on bleeders and the implementation of the overall program took place throughout 1982. The impact on annual and per capita water consumption is depicted in Figure 1 where the impact of the program can be seen commencing in 1983. To put water consumption in the context of how severe a winter may have been the annual freezing degree-days (FDD - the cumulative number of degrees below freezing for each year) is shown relative to the long term mean. A value of 100 would be an average year, 90 would be 90% of the long term average, 110 would be 110% of the long term average, etc.

Annual average water consumption during the winter period (December - April) has been reduced in half based on pre and post 1983 figures for the time period noted on Figure 1. Unaccounted for water which was at 64% of all water produced in 1980 has been as low as 11% since 1983.

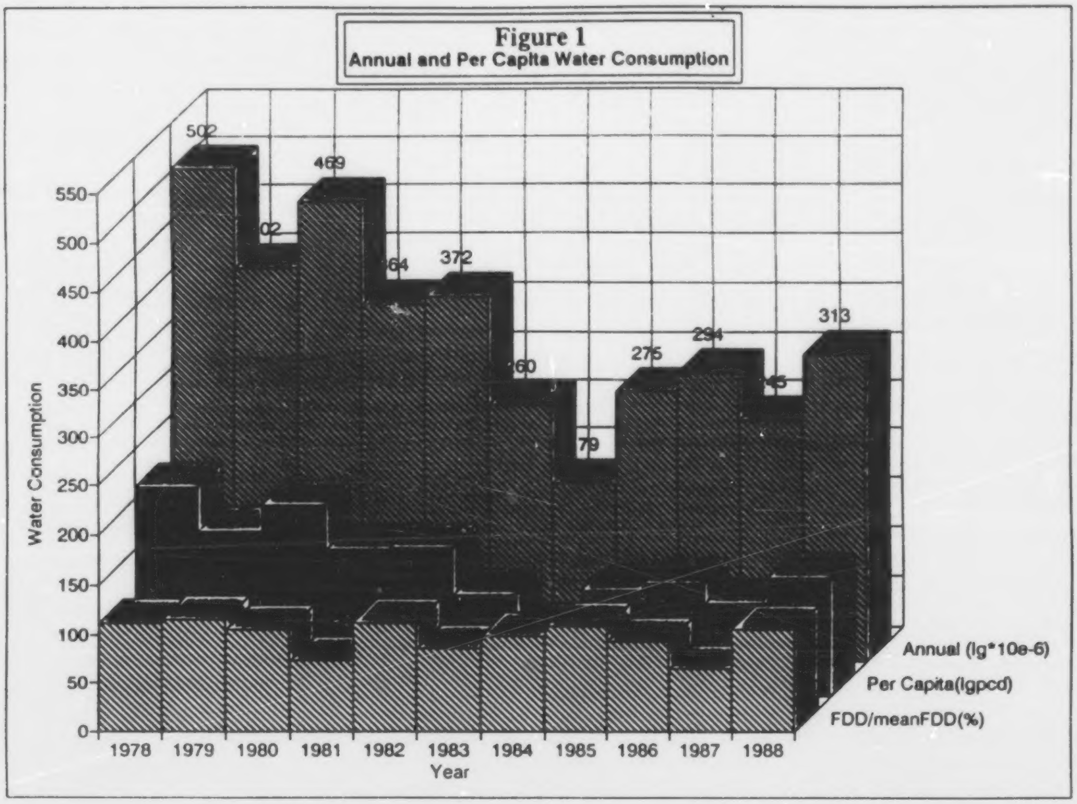

Figure 1
Annual and Per Capita Water Consumption

Table 1

History of Boiler Operation

Winter	Period of Operation	Number of Days Of Operation
74-75	15NOV74-07MAY75	174
75-76	26NOV75-19APR76	146
76-77	12NOV76-30APR77	170
77-78	14NOV77-NOT KNOWN	nr
78-79	NOT KNOWN-21APR79	nr
79-80	08NOV79-22APR80	182
80-81	01DEC80-23APR81	144
81-82	15JAN82-24APR82	100
82-83	31JAN83-26APR83	86
83-84	01FEB84-05APR84	65
84-85	09JAN85-28APR85	110
85-86	21JAN86-28APR86	98
86-87	16JAN87-02MAY87	107
87-88	14JAN88-05MAY88	113
88-89	11JAN89-28APR89	108
89-90	10JAN90-24APR90	105
90-91	16JAN91-24APR91	99

2. **Boiler Operation**

With the implementation of the bleeder control program optimization of boiler operation has been easy to attain. The dramatic drop in water consumption during the winter months has made the same boiler much more efficient in terms of maintaining water temperatures greater than 4.5°C (40°F) at the source.

The provision of temperature probes and the experience gained in determining at what depth of frost penetration will translate to a freezing problem has greatly reduced the length of time the boiler operates annually. The first year the frost probe data was solely used was to initiate operation of the boiler was the winter of 1981-82. The boiler that winter was not put in operation until January 15, 1982 which is approximately 2 months later than was previous standard practice. Table 1 summarizes boiler turn-on/turn-off dates and number of days in operation annually for the time period from 1974-1991. It should be noted that the use of the frost probe data does not appear to have altered the turn-off date from past practice.

3. **Thawing of Municipal Services**

The vast majority of thawing annually is carried out on copper service pipes. The program to develop a safe and reliable method for thawing services accordingly was twofold:

1) Based on records of previously frozen services all services with a past history of freezing were checked during the summer months to ensure that the copper thawing cable could be located and was operable. Services without or with deficient cables were retrofitted so that if the service required thawing the circuit path would be relatively straightforward. This program continues annually.

2) The use of the DC welder was discontinued. The Town purchased a DBH Pipe Thawing Machine - Model RF-82-TA manufactured by Hovey Industries Ltd. The DBH pipe thawing machine is a CSA approved machine specifically designed for electrical thawing of metallic pipes. The machine operates on A.C. rather than D.C., has a computer controlled 2000 ampere A.C. output with the following features:

   - Automatic emergency shutdown in the event of a dangerous circuit condition.
   - Light indicated operator directives and warnings.
   - Automatic output current control to provide safe, efficient operation.

   The DBH pipe thawing machine is much more efficient at thawing than previous experience with the DC welder. It required some time before operators became familiar with the unit, particularly with the automatic shutdown feature. However, frozen services that used to require many hours to thaw are literally thawed in minutes. There has been no occurrences of stray current damage since the DBH machine has been put into operation.

   The capital cost of the machine was $66,000 ($46,000 in 1982 and a $20,000 upgrading in 1988). It was difficult to rationalize at the time of purchase its cost effectiveness in terms of a reduction in time spent thawing services. However, it was believed that continued use of the welder had far too high a potential for eventually costing a human life (eg. due to a fire) and in this context the investment in the DBH machine has been well worth it.

CONCLUSIONS

Through better management, the Town of The Pas has maximized the use of its existing resources. They approach the 1990's with ample hydraulic capacity at the Water Treatment Plant, ample boiler capacity, and deliver a higher and safer level of service to their consumer than appeared to be possible in the early 1980's without a corresponding increase in cost. The deficits in the 1980's are a thing of the past as the water utility now annually generates reserves. The last rate increase for the Town of The Pas water utility was in 1984.

THE WATER AUDIT AND THE ECONOMICS OF LEAKAGE CONTROL

BY

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THE 43RD ANNUAL WESTERN CANADA WATER
& WASTEWATER CONVENTION
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INTRODUCTION

Unfortunately, the time when quality potable water was a cheap and seemingly endless commodity is past. As pollution puts pressure on our fresh water supplies and our underground infrastructure of pipes deteriorate, the cost of transportation from source, treatment and distribution of water are rising dramatically. Water is truly our most precious natural resource. Unfortunately it is also the natural resource most taken for granted by most segments of our society.

Our in-place municipal infrastructures - water mains, sewers and roads - have often not been given the maintenance attention necessary to assure reliability. These necessities tend to be over shadowed by other municipal interests concerned with economic growth, expansion, or social programs since these issues dominate our concepts of progress and are politically secure. Roads have received some attention as potholes tend to jolt our sensibilities from time to time, however, water mains and sewer systems are below ground -out of sight and out of mind. We turn on the tap and expect a constant supply of clear, pure water. We flush the toilet and expect the waste to disappear without a trace. Little thought is given by the general public to the engineering requirements or the construction and maintenance necessary to support such systems safety and reliably.

The sub-surface environment can often be harsh. As we all know it tends to deteriorate our below ground utilities over time. Canada contains extensive supplies of fresh water in river and lake systems and in sub-surface aquifers. These supplies however, are gradually becoming stressed under the pressures of development. Treatment costs are escalating rapidly to meet the quality standards established for potable water by the governing agencies. Supplies of potable water are often pumped considerable distances because local supplies are insufficient or too polluted to purify to acceptable standards. Prairie water supplies for instance are also increasingly under pressure as droughts continue. Many major supplies are endangered not only for consumption purposes but also due to deteriorated water quality. For example, Environment Canada have to date, identified over 360 chemical compounds in the Great Lakes system alone.

Many of the problems associated with water quality and supply require long term solutions. However, many other can be addressed with more immediate programs.

Escalating costs, coupled with ever increasing concern for conservation and environmental protection, are forcing many municipalites and utilities to look more seriously at identifying and reducing their unaccounted-for volumes. This paper looks at this process, with particular emphasis on leakage control and the benefits that can be derived from such a program.

WATER CONSERVATION

Public education and water conservation programs can reduce some of the unnecessary load on our distribution facilities. Water restrictions or the prevention of wasteful usage can be legislated by local municipalities. It is now commonplace to place restrictions on lawn watering during periods of drought. New York City is funding a retrofit program, with public support, to place a restrictive device in shower heads and toilet bowls to lower water consumption in the home. Some electrical utilities are offering rebates to any person who installs a low flow shower head. Their motive is to reduce electricity demand for hot water but reduced water consumption is definitely a fringe benefit.

A feature article with the headline "Stingy Fixtures Will Sip, Not Gulp" appeared on the front page of the January 5, 1990 issue of the London Free Press. The article went on to state: "As Canadians relaize water isn't the inexhaustible resource we've considered it to be, efficient water-saving toilets, taps and shower heads are coming on the market." "Plumbing fixtures that sip rather than gulp water will play a major role in the future...."

Public education and awareness programs can be successful in lowering water consumption during critical periods of shortages. Unfortunately, such programs tend to be short lived as we quickly revert to our wasteful ways when the crisis has passed.

Wastage is also commonplace when customers are not metered but are billed on a flat rate basis. This is the case in many municipalities. Water rates must increase during the next decade to reflect the true value of the commodity and to more equitably distribute costs to the end users. Price is the most effective motive for conservation for most people.

Higher prices are not politically attractive, however the additional revenues will be required if we hope to meet the pressing delivery demands in the years ahead, as well as to upgrade existing distribution systems to acceptable standards.

THE WATER AUDIT

Politicians and the general public are beginning to realize that we can no longer take steady supplies of good quality potable water for granted. Like everything else in this world it has a price.

James MacLaren, the new head of the Ontario Water Commission is predicting a 50% increase in water rates over the next five years. Predictions beyond that time frame have been as high as a 300% increase by the year 2000.

All water systems should be operated as a business with consideration for cost of production and cost of service. It is not uncommon in the water industry to encounter systems with an unaccounted-for percentage of twenty, thirty, forty percent and even higher! No other business could continue to operate with loss factors such as these?

When a municipality or utility suspects a high unaccounted for water situation, it is normal to consider a water audit to ascertain precisly how much water is entering the given system or district and how much of that water is actually being used legitimately.

The unaccounted-for water can be split into different categories. The major ones being:

Meter Error or Malfunction
Inaccurate Record Keeping
Meter Reading Lagtime
Inaccurate Consumption Estimates (non metered systems)
Mains Flushing
Fire Fighting
Municipal Use (Gardens, etc.)
Unauthorized Abstraction From Hydrants (contractors, boat yards & docks, etc.)
Leakage

Leakage on the below ground distribution system is normally the category that far exceeds the others when it comes to the quantity of wastage of treated water.

A complete audit of both the water system operation and administration can quantify to a point and eliminate some of the unaccounted-for figures. A leakage detection and repair program can identify and eliminate much of the rest.

Depending on the extent of the problem and the available budget, the scope of a water audit can vary significantly. Generally there are three main area; an administrative review, flow monitoring, and leakage detection. A complete audit involves all the areas, however each one performed separately can identify and solve certain problems. Water quality is, of course another, but separate concern.

ADMINISTRATIVE REVIEW

The administrative review portion of a water audit consists of a complete review of the so called "office functions" of the Utility as they relate to consumption and supply. This includes such aspects as record keeping, meter reading or estimating procedures, automation and billing.

Poor or inaccurate record keeping can lead to discrepancies in the supplied versus consumed figures. Customers may have services that are not recorded and therefore not billed.

Not keeping track of both the supplied water and the actual consumption can hide problems. For example, if a municipality or utility knows the quantity of water it has received revenues for, but not the amount of water actually treated and supplied, it does not know how much water it is losing. A similiar situation occurs when the quantity of water supplied is known but what is actually being consumed is not. The assumption that all the water supplied is being consumed may not be true. Inaccurate customer counts or non-updated consumption estimates can also be problems.

Meter reading procedures can also lead to discrepancies in unaccounted-for volume figures. The most common problem is lagtime. The utility or municipality may have accurate measurments of the amount of water used in say September. This figure should match the amount of water consumed in September. Due to the meter reading or estimating process the consumption figure may lag by a few weeks to a few months. Generally, these figures balance out over time, but particularly in areas of varing seasonal consumptions, discrepancies can result.

Another source of volume measurement error can be in the automation of the readings. Incorrect data entry, conversion factors (re: imperial to metric) or correction factors that have been preprogrammed in the computer can lead to volume discrepancies and potential lost (or something higher) revenues.

A review of the volume estimates for fire fighting, street cleaning, hydrant flushing, municipal use and other authorized but non-billed uses should also be conducted.

Identification and correction of these types of situations can often result in higher revenues and reduced unaccounted-for volume losses by themselves.

FLOW MONITORING

Accurate flow information is imparitive for all water suppliers. They must ensure that they are not being overbilled for the water they receive and that they are not under (or over) charging for the water they sell.

Flow monitoring can take several forms. The first is through meter checking. Electronic insertion meters or strap-on transient time meters can be installed either upstream or downstream of existing meters to confirm their accuracy. Equipment can also be installed right on the meter to electronically count the revolutions and compaire the readings to the meter output. Generally, this type of equipment provides accuracy in the range of 2-10%. This level of accuracy is sufficient to identify potential problems which can be further checked with more sophisticated testing. It also enables flows over time to be recorded on data loggers and then graphed to show varing flow profiles over the day, the week or the month. Generally, meter checking is done on supply meters, large commercial or industrial meters and a random sample of smaller meters.

Flow monitoring can be used on non-metered systems to confirm estimated consumption or to break out smaller sections of a larger metered system. This gives an indication of actual consumtions where there is no metered information available. It also allows areas of high consumption to be identified. These areas can then be given special attention for attempting to identify such things as leakage, unrecorded customers, large users, or bypassing valves.

Often water flow monitoring is coupled with sewer flow monitoring to get some ideas on the amount, if any, of infiltration into the sewer system.

The information obtained from the flow monitoring can also be used for such things as storage tank sizing requirements, computer network model calibration and for identifying the savings after a leakage control program.

LEAKAGE CONTROL

The third area of a complete water audit is a leakage survey. Generally, leakage control is the single largest factor in reducing unaccounted-for water. Leakage can amount to up to 70% of the unaccounted-for volumes.

The decision to implement a leakage control program is usually based on economics. The cost of the survey and the resulting repairs should be more than offset by the money saved in supply and treatment costs and emergency repairs. This question is one that should be evaluated on a case by case basis, relative to the cost of the product.

For example, a water system that is totally gravity fed with a pure seemingly endless surface supply may not have much incentive to initiate a leakage control program except of course to reduce the costs of reinstatement of roads and property damage and insurance claims due to water main failures. At the other extreme, a system with distant supplies or with a sub-surface supply in which every gallon must be pumped and pumped again coupled with an extensive treatment process would have a much greater incentive to start a leakage control program. The vast majority of water systems, fall somewhere in between these two extremes.

Whatever the situation there is some moral responsibility to eliminate wastage of water and energy.

We must remember that a leak only 1/2 inch in diamter in a water line at a pressure of 60 psi results in a loss of 55.5 Imp. gal/min or 79,920 gal/day. This water must be pumped to a treatment plant, treated, and pumped to the leak. The distribution system itself must be oversized to accommodate this lost water. Excluding the cost of limiting future expansion, if we take the 79,920 gal/day loss and multiply it by a typical rate of $2.00 per thousand gals we get an annual loss of $58,341.60 per year.

There are a number of other benefits that can be realized from an effective leakage control program. These other benefits are numerous but a few are listed below:

1. A reduction in pumping and treatment costs.
2. Elimination of capital borrowing costs by deferral of reinforcement construction.
3. An extension of equipment life at the water treatment plant.
4. A reduction in infiltration into the sewer system and the resulting reduction in demand on the sewer system and the treatment facilities.
5. A reduction in legal and/or insurance expenditures.
6. Improved public relations - fewer breaks, road cuts and shortages.
7. Water main locates and the upgrading of distribution maps.
8. Assistance in budget planning for replacements and additions.
9. Improved employee attitude and morale.
10. Preservation of water for future expansion.
11. Checking for damage after major road or utility reconstruction.

Too often these other benefits are neglected when we evaluate the cost of a leakage control program. These other benefits can be difficult to value but collectively may be worth more than the lost water itself!

LEAKAGE DETECTION TECHNOLOGY

In recent years, through advancements in acoustic instrumentation, microprocessors and improvements in procedures an evloution has occurred in our leakage detection capabilities. It is no longer necessary to wait for roads to cave in, for leaks to surface or for basements to be flooded before leaks can be found. No longer is it necessary to send crews out under adverse weather conditions to conduct emergency repairs to leaks that could have been located and repaired during normal work schedules. With experienced personnel and the proper equipment the majority of water leaks can now be detected, pinpointed and repaired under favorable conditions.

Leakage detection is still, however, not an exact science, even with today's highly sophisticated equipment.

A water leak detection program normally involves some type of sonic evaluation which depends on an operators ability to hear and indentify leak sounds on the water system. Sonic leakage detection is based on the principal that pressurized water leaking through an opening or break in the pipe into an area of atmospheric pressure sets up sounds and vibrations that are audible with special equipment. The majority of water leaks create three distinct sounds.

The first sound, normally in the 500 to 800 Hertz range, originates as an orifice-pipe vibration phenomenon and is transmitted along the pipe wall. This sound is transmitted along the pipe for varying distances depending on pipe material, leak size and system oprating pressures. This vibration can be transmitted considerable distances on metallic pipes but significantly shorter distances on asbestos-cement or plastic pipes. Some of the sound may also be transmitted into the soil surrounding the pipe but the energy is quickly dissipated. This first sound is considered the search sound and is useful in systematically searching an area to determine the presence or absence of leaks. By using highly amplified sonic instruments these sounds can be heard on valves, hydrants, curb valves and other contact points on the system.

The second sound created by a water leak is often referred to as the impact sound. This is caused by the leaking water striking the soil in the leak area.

The third sound is the fountain sound, which is caused by water circulating in water, usually in a cavity in the soil adjacent to the leak. Both the second and third sounds are in the 20 to 300 Hertz range and are generally limited to the immediate area of the leak. Consequently, these sounds are important in pinpointing or verifying the actual leak location.

In the past, utility people depended on water surfacing or by direct listening on the system using a sounding stick or geophone to establish a leak location. Geophones are mechanical devices, similar to a doctors stethoscope, which amplify the volume of leak sounds to audible levels. Modern instruments apply the same principal but amplify the signal electronically to much higher levels. Such instruments are generically classed as sonic type fluid leak detectors. Electronic frequency filters are a standard feature on a number of instruments and effectively filter out unwanted or extraneous frequencies commonly present on water distribution systems.

When a leakage detection program in undertaken, the usual practice is to have two employees (or groups of two) become the leakage crew. The leakage crew moves through the distibution system in a systematic program, listening for the presence of leakage on valves, hydrants, and available access points. When a leak indication is heard, it is marked on the map for further investigation.

Once a leak is detected on the distribution system it then must be pinpointed. Pinpointing is the most difficult aspect of a leakage control program. It is essential that the leakage detection crew have through knowledge and proper training in the use of the equipment to obtain optimum results. In general, the more leaks an operator encounters the more proficient he or she will become at pinpointing the source of the leakage.

During the course of a normal leakage detection program there are other functions that may be performed by the leakage detection crew. Discrepancies such as valves which have been paved over, valve box covers missing, valve boxes filled with dirt or stones, buried curb boxes, etc. can be identified. The leakage crew, during the course of survey will also note discrepanices that may exist on the "working map." For example, they may encounter a hydrant or valve that exists in the field but is not indicated on the map or vice versa. The benefits of more accurate maps can be invaluable.

Approximately 12 years ago the first leak noise correlators appeared on the market. Using the very latest developments in micro-electronics and micro-processor technology, this type of equipment is the most advanced instrument available for accurate leak detection. The result is equipment so effective in its use that few leaks will escape dtection. Heath Consultants leak detection crews use a combination of selective sonics and leak noise correlation techniques, utilizing the Palmer MIcroCorr 4 Correlator. Several municipalities and water utilities are also starting to purchase this more sophisticated equipment.

The MicroCorr, as do other leak noise correlators, uses the technique of leak noise correlation to determine the presence of a leak and then its precise location relative to two sensors placed on the system, (hydrant, main valve, curb box, etc.). The unit is straight forward to use as all the complex mathematics required for signal analysis and correlation are computer controlled, leaving only a few "question and answer" type responses from the operator. By keeping these responses to a minimum and by using very "user friendly" software, the operation of a correlator is very simple even through the calculations involved in the correlation process can be complicated.

An instrument such as the MicroCorr with good contact on a system that has had the correct pipe size, material, length, etc., fed into it can pinpoint a leak to within one inch. The majority of the time the leak is at least within the first hole dug. This ability to accurately pinpoint the leak can be critical on those pipes that are under major roads, across railway tracks, etc.

Effective water leakage detection programs are as much dependent on proper procedures as on good quality instruments. Leak detection personnel must have a knowledge of system design and pipe materials as well as an understanding of the influencing factors to sound attenuation. Soil types, soil resistivity, soil moisture content, pipe materials, pipe size, depth, pipe corrosion and operating pressures all effect the attenutation of sound along the pipe wall and through the soil. Metallic systems such as cast iron and ductile iron require different procedures than do non-metallic systems such as plastic or asbestos-cement. One procedure, or a single instrument, cannont be effective for every condition encountered. For effective results, leak detection personnel must be flexible in their approach and be prepared to adjust procedures depending on the design and field conditions encountered.

SPECIAL SITUATIONS

Situations do occur where conditions are such that leaks cannot be detected by traditional sonic and correlation procedures. They tend to occur more frequently when non-metallic pipe materials are being used. A tracer gas procedure using helium has exhibited a high level of success in locating such leakage over the years. The procedure involves dewatering the section under test and inserting a mixture of 5% to 10% helium in air. Helium is one of the true noble gases being completely inert and non-toxic. It can be used on any type of enclosed system. Due to it's low specific gravity (0.17) and small molecule, helium can vent through very small holes and travel quickly upwards through the soil to the atmosphere.

A specialty instrument developed by Heath Consultants Limited in conjunction with Bell Canada is extremely sensitive to helium. This instrument, called a Heath Gasophon-T, is used to sense the atmosphere at grade directly over the section of piping under test. An operator carrying the instrument walks slowly along the pipe run and the instrument continuously monitors the atmosphere over the pipe. When helium is detected at the surface, the leak location is quickly verified and pinpointed using the various sensitivity settings of the instrument. Helium does not appear naturally so there can be no interference from other sources.

Another method of detecting those difficult to find leaks is through step testing. This is a process where by a flow meter is used to identify the change in flow as various sections of the system are shut off. If a larger than expected drop in flow occurs as a particular section is shut in, it can be due to the isolation of a leak. The search can then be concentrated in this area.

Step testing can be more complicated than tracer gas methods but the system does not have to be dewatered as it does for the tracer gas.

IN HOUSE vs. CONTRACT SERVICE

As was mentioned earlier, today's market has a wide varity of products, such as the Globe Geophones, Heath Son-i-kit and Aqua Scope, and approximately 12 types of Correlators world wide. Prices can range from $35.00 to $80,000.00. When a utility is deciding which approach to take, they have two basic options; contract services or an in-house approach.

Should the utility decide on an in-house program careful evaluation is necessary to insure, that when implemented, the program is an effective one. The selection of equipment for in-house use should include the initial purchase price, repair costs and the cost of providing proper training. The employee's ability to utilize the equipment to its maximum potential is often the difference between an effective program and an ineffective program. Most utilities or municipalities already have the detection equipment and trained personnel who know the system. They may not have the required manpower however.

There are a few companies which offer contract leakage detection services to the water industry. These companies use a variety of instrumentation and methods. Careful evaluation of the potential contractors is advised. This evaluation should include the cost of services, past experience and references, the equipment to be used, the survey procedures, the training and experience of the personnel and the time required (man hours) to complete the contract. Contractors can tend to give more impartial results and because they specialize in the field can be extremely well qualified.

Regardless of the method chosen it is imperative that managment be committed to repairing the leaks once they have been found.

CONCLUSION

Unaccounted for water is a problem common in the water industry. As infrastructures deteriorate, quality supplies dwindle, operating costs increase and demand rises, this problem is becoming more and more critical. Many municipalities and utilities have already realized the benefits of the water system audit. Many have had comprehensive leakage control programs in place for many years. Many have not! All water suppliers, will, at some point in time, be forced to address this problem and institute solutions that are both practical and cost effective. Fortunately, equipment and expertise are available to the modern day water utility to provide a logical solution to this problem at an affordable price.

Man is an innovative and creative creature. We can maintain and ensure a constant supply of safe potable water for future generations if we desire. Short term measures include leakage control and conservation as well as realistic pricing to provide revenues for proper maintenance and quality control programs. Long term measures must address the deteriorated infrastructures and the environment and aim at reducing pollution and restoring presently contaminated sources to acceptable health levels. May we not experience the lament of Samuel Taylor Coleridge in the Rime of the Ancient Mariner, "Water, water everywhere, nor any drop to drink."

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POLYMORPHIC RESIN COATINGS FOR USE IN WASTE WATER HANDLING FACILITIES

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INTRODUCTION

Concrete construction materials used in waste water handling facilities are vulnerable to chemical attack. Waste water itself generally has little corrosive effect on concrete. However, in some situations, hydrogen sulphide (H_2S) gas is generated and released in the sewer environment. Combined with oxygen and the high humidity generally present in this environment, the H_2S gas reacts to form H_2SO_4 which is extremely detrimental to concrete surfaces.

Over the years, many types of epoxy, urethane, cementitious, and other polymer coating systems have been used for the protection of concrete in waste water handling facilities. These coatings have met with various degrees of success.

This paper will examine the topic from the practical/application experience gained in the field.

THE DETERIORATION PROBLEM

Hydrogen Sulfide (H_2S) is produced when sewage becomes anaerobic within the sewer. Anaerobic conditions generally result from stagnant flow or very long transport times. Hydrogen Sulphide generation is also highly dependant on the presence of sulphates in the water supply, and the temperature of the sewage itself.

When hydrogen sulphide is produced, it is generally dissolved in solution. Dissolved in solution, hydrogen sulphide does very little damage to concrete structures. The problem begins when the H_2S is driven out of solution by turbulence, aeration, or other method. In the air, the H_2S gas reacts with oxygen and moisture present to form a sulphuric acid condensate on exposed concrete surfaces. The very low pH of this condensate is extremely aggressive towards concrete surfaces. The reaction is primarily with the cement paste and rarely are the aggregates attacked. The resultant surface is therefore eroded similar to exposed aggregate, varying in degree with the level of attack.

SURFACE PREPARATION

With all types of coatings, surface preparation remains the most critical phase of application. Without adequate surface preparation not even the most resistant coatings will perform satisfactorily in the long term. A good bond and a sound substrate is essential for a long-term installation.

Generally a thorough sandblast followed by either vacuuming, air blow down, or water wash down to remove dust from the prepared surface is satisfactory. The amount of surface moisture tolerable during coating application is highly dependant on the specific coating material. The overall project schedule and drying conditions may limit the method of surface preparation (ie. water wash down) and may even restrict the type of coating material suitable for a given application.

Sufficient surface preparation can be tested by a surface pH test and, a tensile bond pull test if applicable. It is very important to test the surface pH after sandblasting.

The surface pH test will reveal if there is any contaminated concrete remaining on the prepared surface. Other contaminated concrete can be soft and deteriorated on the surface with contaminated sound concrete underneath. It is important that all of the contaminated sound concrete is also removed. If it is not, the coating will adhere and perform satisfactorily in the short term, but will fail in the long term, since the contaminated concrete will continue to deteriorate underneath the applied coating. Performing the surface pH testing will very quickly identify the existence of contaminated concrete on the surface and eliminate many long-term coating failures.

After proper surface preparation, the prepared surface will have an open, exposed aggregate appearance. The surface will be much rougher than a prepared steel surface, and will have many small holes and air bubbles. From experience we have found the application of a coating alone to be unsatisfactory for this type of surface. It can be done, but it is very difficult to coat and seal the hundreds of these holes which exist in a surface. If even one or two are not completely covered, the coating will be subject to failure due to chemical attack of the concrete and undermining of the coating itself.

The preferred option therefore is to apply a surface parge coat to the entire surface prior to coating application. This is a relatively simple procedure which will significantly extend the life of the coating.

The installed cost of the coating will be increased by the parging application, but the effectiveness and service life of the coating will be extended to more than offset the additional cost.

TYPES OF COATINGS

Many types of coatings have been used to address hydrogen sulphide attack on concrete in sewage treatment and transfer facilities. Cementitious, coal tar epoxy, amine epoxy, urethane, polyester, and polymorphic resin coatings have all been used with various degrees of success.

The coatings classification types (epoxy, urethane, etc.) are generic classifications. It is important to note that these polymer materials can be formulated to have very wide ranging physical properties, even within a generic classification type. It is therefore possible to formulate a single polymer type such as an epoxy for example to have properties ranging from semi-flexible to rigid with varying degrees of chemical resistance. The discussion which follows is based on the general properties of a polymer resin classification.

Cementitious Coatings

Cementitious coatings were probably the first coatings utilized in sewage treatment facilities. Cementitious coatings are widely used in the waterproofing industry. Unfortunately, cementitious coatings do not have sufficient chemical resistance to withstand the acid attack caused by hydrogen sulphide gas. This is intuitively obvious, since the cement paste is chemically attacked by the acidic conditions.

Coal Tar Epoxy Coatings

Coal tar epoxy coatings have been used for many years for the protection of both steel and concrete components in waste water treatment facilities. The performance of these coatings have been relatively good, but environmental concerns have virtually eliminated the use of this type of coating material.

Amine Epoxy Coatings

Amine epoxy coatings are probably the most widely accepted general purpose coatings in use today. Amine epoxy coatings are used in many types of industries, and for many types of chemical service applications. Performance in wastewater applications has been satisfactory in many applications, but disappointing in others. Other than inadequate surface preparation, the main causes of failure have been chemical attack of the coating material itself and failures due to pinholes in the coatings.

Urethane Coatings

Urethane coatings have not been widely used in waste water applications. In theory, urethane coatings possess the advantage of being flexible and therefore able to "move" with the structure. In practice, urethane coatings have experienced many failures due to pinholes and bubbling. The high degree of bubbling is due to the water reactive nature of uncured urethane resins. If the uncured urethane coating material comes in contact with water, a chemical reaction will occur. Generally, this reaction results in an improperly cured urethane coating and the formation and release of gas bubbles.

Urethane coating manufacturers have been working on formulations and the addition of inhibitors/defoaming agents to minimize this problem.

Polyester Coatings

Polyester coatings possess good resistance to acid attack. As such, polyester coatings are well suited to the chemical environment in waste water treatment facilities.

Despite this advantage, polyester coatings have not been widely used, primarily because of their intolerance for damp surfaces, and the potential effects of saponification. Saponification is the softening (or "soapifying") of the surface of a polyester coating on exposure to strong alkalies. This is a potential risk at the bond line since good concrete, particularly fresh concrete is a highly alkaline material.

Polymorphic Resin Coatings

Polymorphic resin coatings are essentially modified polyester coatings. The primary modification is in the degree of cross linking in the hardened coating. Greater cross linking increases both the mechanical and chemical properties of the resin. As compared to polyester coatings, polymorphic resin coatings have higher tensile and compressive strengths. Chemical resistance is also improved, particularly for strong acids and border line chemical environments.

The chemical resistance and durability of polymorphic resin coatings has been demonstrated by accelerated tests and field trials. The City of Los Angeles Sanitation Department initiated a large scale accelerated test program to evaluate a very wide cross section of available coatings for application in protecting the severely deteriorated Los Angeles infrastructure.

The test environment involved the preparation and application of the coatings to basins which were comprised of both new and acid attacked concrete surfaces. After preparation and application of the coating, the basin is exposed to an acid environment designed to test the installed coating in an accelerated manner. Sketches detailing the test procedure are shown in figures 1 through 4. Under this accelerated test, a polymorphic resin coating was the only coating to receive a #1 rating in all categories.

Near the end of the Los Angeles Sanitation Department test program, the Greater Vancouver Regional District (GVRD) recognized that a serious concrete deterioration problem caused by H_2S gas generation existed within Vancouver and its surrounding districts. Instead of going to the lab to develop a test program, the GVRD went to the field. The GVRD located a section which was experiencing sever H_2S gas related deterioration. In this chamber, the GVRD applied sections of a large number of available coatings. Many different types of coatings were applied.

The area was re-examined after a period of time (slightly less than one year). The field test confirmed the laboratory test results determined for polymorphic resin coatings. The polymorphic resin coating applied by the GVRD was the only coating in the test program which revealed no deterioration or failure during the test period.

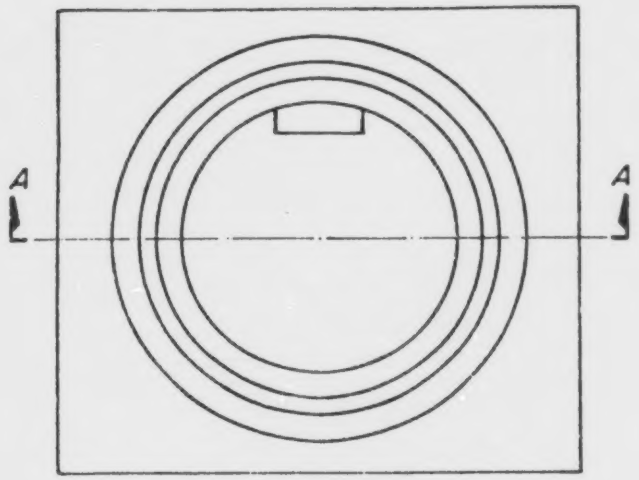
Similar results have been found in numerous field applications. A number of the projects utilizing polymorphic resin coatings have even been in applications where other types of coatings have failed.

In addition to chemical resistance and good mechanical properties, polymorphic resin coatings have some application characteristics which help to assure the long term performance in waste water applications.

The application characteristic which is most beneficial is a variable cure rate. A polymorphic resin coating may be accelerated through the use of an accelerator, and a fast cure rate can be very beneficial in waste water applications.

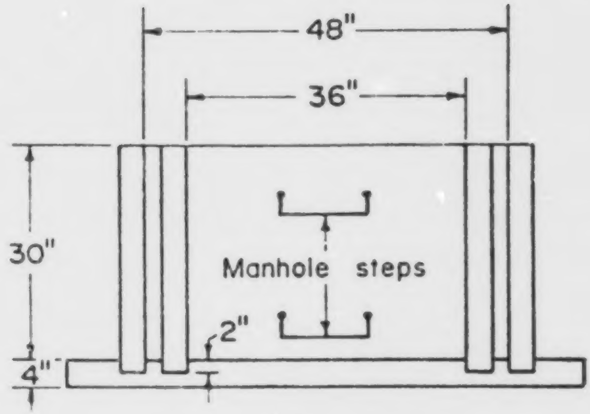
A fast cure rate is beneficial, particularly in restoration applications since the concrete is often in a saturated/wet condition. The surface of the concrete can be dried relatively easily, but it is virtually impossible to dry a mass of concrete in a reasonable period of time. As a result there is often a vapour pressure present during coating application. If the coating material cures slowly, there is a much greater likelihood of pinholes developing in the coating prior to full cure. With a very fast cure rate, the likelihood of developing pinholes due to moisture vapour pressure is greatly reduced.

In addition to a fast cure rate, a multiple coat application also reduces the incidence of pinholes to a great extent. For most applications, a three coat (three layer) application is employed. From experience, a three coat application has been satisfactory in most situations to prevent pinhole related failures.



PLAN OF TEST TANK

Fig. 1



SECTION A-A

Fig. 2

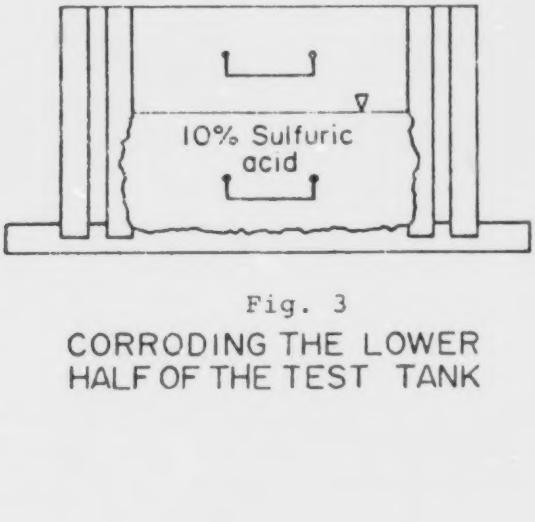


Fig. 3

CORRODING THE LOWER HALF OF THE TEST TANK

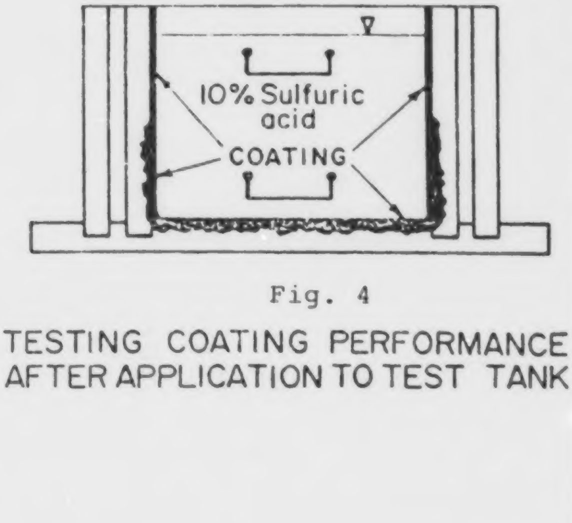


Fig. 4

TESTING COATING PERFORMANCE AFTER APPLICATION TO TEST TANK

The only disadvantage to polymorphic resin coatings encountered to date has been the need for specialized two-component spray equipment which is required for most applications of any size. The reason for this is that polymorphic resin coatings harden very quickly, and the two component equipment is required for proper application.

CONCLUSIONS

Proper surface preparation is one of the most important factors in the success or failure of concrete coatings. Many types of coatings including cementitious, coal tar epoxy, amine epoxy, urethane, polyester, and polymorphic resin coatings have been used in waste water applications subject to H_2S gas attack with varying degrees of success.

Polymorphic resin coatings are well suited to waste water (H_2S gas) applications. Polymorphic resin coatings have excellent mechanical and chemical resistant properties which stand up to hydrogen sulphide/sulphuric acid attack. In addition, a fast cure rate combined with a multiple coat application virtually eliminates the risk of pinholes and pinhole related coating failure. Specialized equipment is usually required to properly apply polymorphic resin coatings.

SULPHUR DIOXIDE DECHLORINATION OF WASTEWATER

by

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INTRODUCTION

Chlorine was first used in potable water treatment about the turn of the century (White, 1986), and was used for disinfecting sewage effluents as early as the late 1800's (White, 1978). In the decades that followed, chlorine use became widespread due to its powerful disinfectant capabilities, availability, ease of application, and relatively low cost (Ward and DeGraeve, 1978b).

Although alternative methods of disinfection have been developed, no other chemical has been able to match chlorine for its potency and wide range of effectiveness (White, 1986).
Chlorine is applied in two different ways; as chlorine gas, or as a hypochlorite solution (usually sodium hypochlorite).

WASTEWATER DISINFECTION

Issues

The practice of disinfecting municipal wastewater creates a dilemma which has sparked continued debate (Environment Canada, 1978; Thompson et. al., 1989).
The driving force behind wastewater disinfection is, and always has been, the protection of public health. Wastewaters are disinfected before discharge to control the incidence of waterborne disease transmitted through ingestion, recreational contact, or consumption of shellfish.

Chlorination is the most common method of disinfection used (Shertzer, 1986). The debate arises because the residual chlorine in the effluent is toxic to aquatic life. A trade-off, therefore, must be made between the protection of public health and the protection of aquatic life. While the protection of public health has always taken priority, the need for protecting aquatic life is being given an increased emphasis (Thompson et. al., 1989).

Regulations in North America vary throughout the provinces and states (Buxton and Ross, 1979). Requirements may call for continuous chlorination, seasonal chlorination, chlorination as judged on a case by case basis, disinfection by some other method of choice, or no disinfection at all (Environment Canada, 1978). When chlorination is employed, there are different requirements for residual chlorine levels (Environment Canada, 1978).

Chlorination of Wastewater

When chlorine gas is dissolved in water, it undergoes a hydrolysis reaction to produce hypochlorous acid:

$$Cl_2 + H_2O \rightarrow HOCl + HCl \quad (1)$$

This reaction occurs very quickly (White, 1986). Hypochlorous acid is a weak acid and will partially dissociate:

$$HOCl \rightleftharpoons H^+ + OCl^- \quad (2)$$

The degree of dissociation is dependent on the pH of the solution. As the pH drops, this reaction is forced to the left resulting in a higher percentage of HOCl and a lower amount of OCl^-.

If sodium hypochlorite is used, a similar reaction occurs resulting in the same active chlorine species:

$$NaOCl + H_2O \rightarrow HOCl + NaOH \quad (3)$$

The key difference between using chlorine gas or sodium hypochlorite is in the resulting pH. The use of chlorine gas results in the production of hydrochloric acid, whereas the used of sodium hypochlorite is accompanied by addition of base. This affects the relative amounts of HOCl and OCl^- in solution.

The expression 'free chlorine' refers to the chlorine present as either HOCl or OCl^-. 'Combined chlorine' is the result of the reaction between chlorine and ammonia. The resulting formation of chloramines is very common as ammonia is present in all but a few wastewater streams (White, 1986). These reactions are:

$$Cl_2 + NH_3 \rightarrow NH_2Cl + HCl \quad (4)$$

$$2Cl_2 + NH_3 \rightarrow NHCl_2 + 2HCl \quad (5)$$

$$3Cl_2 + NH_3 \rightarrow NCl_3 + 3HCl \quad (6)$$

The relative amounts of monochloramine, dichloramine and trichloramine are dependant on several factors including the pH of the wastewater, the ratio of ammonia to free chlorine, and the concentrations of ammonia and chlorine. Reaction 6 only occurs when the chlorine to ammonia nitrogen ratio is very high, or if the pH of the solution is below 5 (White, 1986).

TOXICITY OF CHLORINATED AND DECHLORINATED EFFLUENTS

Toxic Effects

The toxic effects of residual chlorine to various fish and plants has been studied by many researchers.

Brungs (1973), in a literature review, found several variables to be important with regard to chlorine toxicity: pH, species of aquatic life, relative concentrations of free chorine and chloramines, and length of exposure.

In another review, Brooks and Seegert (1975) additionally concluded that the life stage and size of an organism is an important factor. Water temperature was also cited as being a factor for some species.

In some extensive studies, Basch and Truchan (1973) determined safe concentrations of residual chlorine for various types of fish. They generated numbers for both continous and intermittent chorination for coldwater and warmwater intolerant species. The most stringent concentration limit was 0.005 mg/L to protect coldwater intolerant species during continuous exposure.

Middaugh et. al. (1977) studied the effects of chlorination on early life stages of striped bass. When exposed to 0.01 mg/L total residual chlorine they found a greater than 50% reduction in the number of embryos that hatched. Many others (Arthur et. al., 1975; Morgan and Prince, 1978; Roberts, 1980; Ward and DeGraeve, 1978a, 1978b, 1978c) have also shown negative consequences of residual chlorine to aquatic life.

Much work has also been done to study the effects on aquatic life of dechlorinated effluents, and the dechlorinating chemicals. Collins and Deaner (1973) concluded that an effluent that has been chlorinated and dechlorinated is less toxic than an unchlorinated one.

Hall et. al. (1981) studied the toxic effects of chlorine and sulphur dioxide on the eggs and larvae of striped bass. They also studied the toxicity of an effluent dechlorinated with sulphur dioxide.

For continuous exposure for 36 hours, their results showed that chlorine was lethal to bass eggs at concentrations between 0.06 mg/L and 2.0 mg/L. When exposed to the same concentrations of SO_2, the increase above natural mortality was marginal at 6%. As well, mortality did not increase with increasing concentration.

When exposed to effluents that had been chlorinated and subsequently dechlorinated, the egg mortality was greatly reduced to an average of 11% above natural mortality.
Similar results were obtained when bass larvae were exposed for 96 hours. For the dechlorinated effluent, mortality averaged 22% above natural.

The authors commented that sulfite residuals tend to dissipate quickly, and that concentrations of SO_2 used for dechlorination would likely have minimal effects. They also suggested that the increases in mortality may have been caused by the formation of chlorinated organics.

Another area of concern are the nonlethal effects of residual chlorine. For instance, fish species may avoid water containing chlorine residuals. This may have negative implications such as reducing the area in which the fish can find food, or interfering with spawning paths (Thompson et.al., 1989).

Solutions

In situations where aquatic life is threatened as a result of chlorination of wastewater, there are three general strategies that can be employed.

One is to not disinfect the wastewater. Depending on the applicable regulations, the nature of the receiving body of water, and the downstream uses of the water, this may or may not be an option.

A second option is to reduce the amount of chlorine used or to use another disinfectant. Since the push to chlorinate wastewater has been for the protection of public health, many wastewater effluents may be overchlorinated (Connery et. al., 1989). Potential alternatives to chlorine for disinfection include ozone, UV radiation, bromine chloride, and chlorine dioxide.

A third option used to combat the toxicity of a chlorinated effluent is to dechlorinate it. This can be accomplished in several ways, the most common of which is chemical dechlorination using sulphur dioxide. In fact, there are over 500 wastewater treatment plants in North America employing chemical dechlorination (WPCF, 1987), with sulphur dioxide being overwhelmingly preferred (Helz and Kosak-Channing, 1984).

SO_2 DECHLORINATION

Dechlorination Reactions

Similar to the reaction of chlorine in water, sulphur dioxide dissolves in water to form sulphurous acid.

$$SO_2 + H_2O \rightarrow H_2SO_3 \quad (7)$$

This is a relatively easy step as SO_2 is quite soluble in water. Sulphurous acid is a reducing agent and can react with excess chlorine, a strong oxidizer. Dechlorination of free chlorine produces sulphuric and hydrochloric acid:

$$H_2SO_3 + HOCl \rightarrow H_2SO_4 + HCl \quad (8)$$

Sulphur dioxide works equally well in dechlorinating all forms of combined chlorine to form ammonium chloride, sulphuric acid, and hydrochloric acid:

$$H_2SO_3 + NH_2Cl + H_2O \rightarrow NH_4Cl + H_2SO_4 \quad (9)$$
$$2H_2SO_3 + NHCl_2 + 2H_2O \rightarrow NH_4Cl + 2H_2SO_4 + HCl \quad (10)$$
$$3H_2SO_3 + NCl_3 + 3H_2O \rightarrow NH_4Cl + 3H_2SO_4 + 2HCl \quad (11)$$

These dechlorination reactions are essentially instantaneous, taking only a few seconds to complete (White, 1986). This rules out any need for a special mixing tank. The only essential requirment is for good mixing of the sulphur dioxide in the effluent to assure complete SO_2-Cl_2 contact.

Dosage of SO_2

To determine the quantity of sulphur dioxide required for dechlorination, it is convenient to combine the individual steps to find the net reaction. Consider the equations for the addition of chlorine, the addition of sulphur dioxide, and the dechlorination of free chlorine. When added together, the result is a single step representation of the dechlorination process:

$$Cl_2 + H_2O \rightarrow HOCl + HCl \quad (1)$$
$$SO_2 + H_2O \rightarrow H_2SO_3 \quad (7)$$
$$\underline{H_2SO_3 + HOCl \rightarrow H_2SO_4 + HCl} \quad (8)$$
$$Cl_2 + SO_2 + 2H_2O \rightarrow H_2SO_4 + 2HCl \quad (12)$$

It can be seen from this that one mole of sulphur dioxide is needed for every mole of chlorine (as Cl_2). On a weight basis, this equates to 0.9 grams of SO_2 for every gram of Cl_2.

In practice, sulphur dioxide dechlorination systems are usually designed to be able to give a dose as high as 2:1. This allows for expansion, ensures full dechlorination can be achieved in the case of chlorine overdose, and gives the system flexibility (Constable, 1979).

Drawbacks

Sulphur dioxide dechlorination technology is generally well known and easy to apply. But there are two potential drawbacks to be considered.

First, all the dechlorination reactions (equations 8,9,10,11) produce acids as products. This presents the potential for lowering of pH in the effluent. However, the alkalinity in most wastewaters is sufficient to combat the dose of SO_2 used in dechlorination (Helz and Kosak-Channing, 1984). The lowering of effluent pH is not normally seen in practice (Gan et. al., 1979).

Second, since some degree of overdosing of sulphur dioxide is usually practiced, there is often excess sulphurous acid in the water. Concern arises because sulphurous acid can react with dissolved oxygen to form sulphuric acid:

$$2H_2SO_3 + O_2 \rightarrow 2H_2SO_4 \quad (13)$$

Again, this is not normally a problem. The mixing that occurs when the sulphur dioxide is first applied usually results in sufficient air/water contact to supply the needed oxygen to overcome this demand. In fact, one study (Gan et. al., 1979) found dissolved oxygen to be higher in the dechlorinated effluent than before chlorination.

Further to this, reaction 13 probably proceeds too slowly to be of great concern (Helz and Kosak-Channing, 1984). Therefore, despite the fact that SO_2 can add an oxygen demand to the water, reaeration at the end of the process is rarely required (Chen and Gan, 1981).

SO_2 DECHLORINATION SYSTEM

System Components

The set-up and operation of a dechlorination system is very much like a chlorination system. In fact, the equipment used to feed sulphur dioxide is almost identical to the equipment used to feed chlorine (Aschoff et. al., 1984). The main difference is in materials of construction: chlorine makes use of monel and ABS, while sulphur dioxide makes use of SS316 and PVC (White, 1986; Constable, 1979).

Sizing of storage facilities and delivery equipment is made easy by the fact that many of the properties (especially density and viscosity) of sulphur dioxide are close to those of chlorine (White, 1986).

Although there are a variety of ways in which a sulphur dioxide dechlorination system can be configured, there are four basic components common to all systems (White, 1986):

1. SO_2 supply
2. vacuum regulator
3. SO_2 gas flow control
4. ejector/sulphonator

Sulphur dioxide is supplied by ton containers or in bulk by tank truck or rail. After the SO_2 is withdrawn and dissolved in water, the sulphurous acid solution is fed through a diffuser into the effluent to be dechlorinated. Good mixing of the solution can be accomplished by ensuring turbulent flow, using mechanical mixing, or by other means such as hydraulic breaks and jumps (Constable, 1979).

Control Strategies

Process control has generally been one of the more difficult aspects of SO_2 dechlorination. In addition to measuring effluent flowrate, control of a sulphur dioxide dechlorination system usually requires monitoring the concentration o either residual chlorine or sulphur dioxide. Over the years, the problem with this has been that techniques and instrumentation to measure a zero chlorine residual have been weak, and for SO_2 have been essentially non-existent (Finger et. al., 1985).

Although many variations exist, there are four basic types of control systems used in dechlorination (Constable, 1979):

1. flow proportioning
2. residual, or feedback
3. compound loop
4. feed forward

Flow Proportioning

Flow proportioning control is used when the chlorine residual is essentially constant and the flow is the on parameter that changes (Constable, 1979). This system (Figure 1) takes a signal from a flow meter and us it to adjust an automatic valve controlling SO_2 feed. This method of control suffers because there is no tes on how well dechlorination is being done as the effluent is not monitored (Finger et. al., 1985).

Residual, or Feedback

Residual control (Figure 2), as the name implies, uses the chlorine residual in the final effluent to control addition of sulphur dioxide. By itself, this control system is used in situations where flowrate is constant but chlorine demand varies (Constable, 1979). The method suffers because instrument calibration with a zero chlorine residual is very difficult to maintain due to electrode polarization (White, 1986; Constable, 1979).

To some degree, this problem can be overcome by adding some chlorinated water of known concentration (bias water) to the effluent being analysed (Adams et. al, 1983). Mixing these two streams guarantees a positive non-zero chlorine residual in the sample actually being analysed (Adams et. al, 1983). The weakness of this system is that both the effluent sample flow and the bias water flow must be kept consta and this requires a high degree of accuracy for a plant environment (Finger et. al., 1985).

New instrumentation is now available that employs an internal biasing system. This makes it easier to monitor and maintain a zero chlorine residual, or in fact, an SO_2 overdose.

Compound Loop

Flow proportioning and residual control are examples of simple feedforward and feedback control signals respectively. A compound loop system (Figure 3) is a combination of these two (Constable, 1979, Witkowski et. al., 1980). This type of control has the benefit of using two measurements which minimizes cycling due to variations in either flow or chlorine residual (Constable, 1979). It still suffers, however, from the biasing problem in chlorine residual measurement.

Feed Forward

Feed forward control (Figure 4) is the method used by the vast majority of plants utilizing sulphur dioxide for dechlorination (Gan et. al., 1979). In feedforward control, a flow signal is combined with a residual measurement taken before the addition of SO_2 (Finger et. al., 1985). This gives the chlorine discharge rate and allows the sulphur dioxide feedrate to be controlled appropriately (Finger et. al., 1985).

This type of system has many advantages. Like the compound loop control system, it uses both a flow signal and a residual measurement. It does not, however, suffer from calibration drift, since it measures a non-zero chlorine residual (Finger et. al., 1985). No biasing of the sample is required.

The one downfall to this system is that it does not lend itself to a performance check; there is no post dechlorination monitoring of residual chlorine (Finger et. al., 1985).

OTHER METHODS OF DECHLORINATION

There are basically four other dechlorinating methods available that deserve mention; sulphur compounds other than SO_2, hydrogen peroxide, activated carbon, and holding ponds. In most cases, they are less attractive alternatives to sulphur dioxide for either technical or economic reasons.

Sulphur Compounds

Other than sulphur dioxide, this group includes sodium sulphite, sodium bisulphite, sodium metabisulphite, and sodium thiosulphate.

In principle, these chemicals all work in the same way. When dissolved in water, each of these compounds will produce the sulphite ion (SO_3^-). The actual dechlorination reactions are then very similar to those of sulphur dioxide. On a weight basis, a greater quantity of these chemicals is required for stoichiometric dechlorination compared with the 0.9 kg/kg of sulphur dioxide.

Sodium sulphite has the disadvantage of being a solid which makes it harder to handle and apply. On a weight basis, the sodium sulphite to chlorine dose is 1.78 kg/kg. This works out to a chemical cost of about five times greater than SO_2 at current prices.
Sodium bisulphite and sodium metabsulphite are both applied as solutions producing sodium bisulphate and hydrochloric acid in the dechlorination reaction. In each case, the chemical cost is about five times that of sulphur dioxide.

Sodium thiosulphate is used as a dechlorinating chemical in laboratory work only. Its reaction with chlorine is relatively slow compared with other sulphur compounds (White, 1986).

Hydrogen Peroxide

Hydrogen peroxide suffers the great disadvantage of being ineffective against combined chlorine (Harrison et. al., 1975). This effectively rules it out as an alternative for most wastewater dechlorination needs. On a chemical cost basis, it is about five times more expensive than sulphur dioxide.

Activated carbon

Activated carbon can be used for dechlorination, but there are problems with this method. It tends not to be very effective at removing chloramines, and at best, does so slowly (White, 1986).

Activated carbon will also absorb other organic contaminants concurrently with dechlorination. This is not always desired. Depending on the contaminants in question, this may also reduce the dechlorinating efficiency of the carbon.

Holding Pond

Holding ponds work by exposing chlorine residual to UV light and allowing it to decompose by time. Technically, this method is slow with free chlorine, and ineffective for chloramines. Economically, one of the major drawbacks of holding ponds is the cost of the land required (White, 1986).

SUMMARY

Wastewater disinfection is widely practiced for the protection of public health. Due to its effectiveness, low cost and ease of application, chlorine has evolved as the primary disinfectant used. Concern is now focussed on discharging treated wastewater because of residual chlorine toxicity to fish.

Dechlorination by sulphur dioxide provides an effective and economical solution to this problem. Chlorinated wastewaters that have been dechlorinated with SO_2 have shown a greatly reduced toxicity toward aquatic life. Application of sulphur dioxide is similar to chlorine, making it relatively easy to use.

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LARGE DIAMETER PIPE CLEANING PROBLEMS AND SOLUTIONS

by

Don Derlein
Live line Ltd

Cities across both of our nations are facing severe unseen problems. Our large diameter interceptor sewer systems are quietly deteriorating. I say quietly, because unlike our roads, bridges and buildings, which are quite visible and accessible to evaluate and repair, the sewer system is out of sight and unfortunately out of the public's mind. Funding for the visible problems is much easier to sell to the taxpayers than are the underground defects that not only can't be seen, but most often are not understood by the general public. "My toilet still works... what is the problem?" Part of this dilemma is a result of the general publics lack of understanding of the workings of a sewer system. We all understand that roads and bridges may only have a useful life of so many years but we forget that pipes also have a useful life. Unfortunately we are reminded of this fact when a large section of road collapses into a failed section of sewer pipe.

We as contractors, engineers and public officials must put forth the time and money needed to educate the general public on the importance of maintaining and upgrading our sewer and water systems. This could possibly be done through contractor associations, brochures mailed with water bills, television ads, presentations to school and civic groups. Now it seems the education the public is given is in the form of higher user rates and taxes. This method is a negative approach to a problem. One thing that we must keep in mind is that a "confused mind always says NO", so we must help the public understand this problem because it will only get worse.

How did we get all these sewer problems anyway? In my opinion, the major culprit is money, or more specific, the lack of it! We have pipelines that have been in the ground way longer than the designers ever intended, but there is not enough money to replace them. Some of these pipelines have so many other improvements on top of them that the replacement is cost prohibitive. Some of the other contributing factors to our problems are, poor initial installation, old style joint material allowing root and ground water infiltration, inadequate maintenance which allows for sediment build up. This sediment buildup can lead to high levels of hydrogen sulfide and that will cause the breakdown of the lime in concrete pipe. Lines that are laid with slopes that do not allow the proper velocity for self cleaning are very prone to hydrogen sulfide damage.

There are some cities that have a very effective cleaning program. This program involves a schedule that gets all the sewers at least jetted every three years. To get this program started may require heavy cleaning in the interceptor system. The initial cost to implement this program is quite high but the per foot cost to maintain a clean system is relatively inexpensive. A clean system allows you to do television evaluation much easier than having to fight through each individual line. Sewers without large amounts of sediment build-up can also be more accurately monitored as to flow capacity. Perhaps the greatest advantage to a cleaning program is the one that may be the most difficult to put a dollar value on. That is, the deterioration of the lines, the collapses and subsequent disruptions. This includes sewerage backup and public damage. This is one of the times when the general public wonders why something wasn't done to prevent this mess. Granted, a positive maintenance program may not prevent collapses and the like but it is way ahead of whatever is in second place.

OK! So I haven't told you anything that you didn't already know! Chances are, if you didn't have problems in your system you would be over at the hotel sipping on a cool one. You are here...you do have problems...so let's get to the meat of this presentation. Let me tell you about "Live Line".

I am a sewer contractor and have been in public works sewer construction for 22 years. My company is part of a group of companies that make up the third largest specialty utility contractor in the United States, according to "Engineering News Record 1990". We have worked primarily with municipalities so we understand and sometimes sympathize with the problems associated with government bureaucracies. We are specialists in Direct Bury Pipelines, Pump Station Installation (both water and sewerage), Structural Concrete, and Rehabilitation of all of the above.

The development of the Live Line process was a result of our involvement in large diameter rehabilitation, more specifically, sliplining. The companies that were doing the cleaning for us were faced with many problems. Some of the major difficulties were:

1) Trying to clean with the heavy flows.
2) Moving vast amounts of sediment found in these lines.
3) Using huge quantities of water for the jets.
4) Most importantly, getting the line positively clean to allow us to push the liner without getting stuck.

Watching sewer cleaners struggle with these problems prompted us to put a lot of engineering and money into what we call the "Live Line" process.

This process involves some complicated machinery that in effect, dredges a sewer line. The unit that contains the pumping mechanism is called the "Hydro Hog", this unit is pulled through the pipe with a computer controlled winch. The larger debris is captured in the first part of this unit and eventually is brought to the next manhole for removal. The sand, gravel muck and other debris pass through the screen and are pumped to the surface. The computer is given the line size and average silt depth. This provides the necessary information to compute the pulling speed. This pulling speed is one of the most important aspects of the cleaning process. If we pull too slow, production suffers and if we pull to fast, we may overload the system and risk plugging the discharge hoses.

The sludge and water mixture is pumped to the surface and passes through a two stage separator unit. The first stage removes the larger particles and floatables. The balance of the mixture goes through a centrifugal separator that removes the water from the smaller grit. The water is returned to the sewer and the debris is kept circulating in this tank until the shuttle truck arrives to pick up another load to transport to your disposal site. All of this is accomplished within a closed system.

All of the "Live Line" equipment is designed to be completely portable and easily transported to your jobsite. Each piece of equipment has its own lighting system if traffic or other considerations require night operation.

What about the cost of this service? Cost will vary based on the following variables:

1) Travel miles to jobsite.
2) Total footage to be cleaned.
3) Depth of Debris.
4) Size of Line.
5) Manhole accessibility.
6) Length of individual runs.
7) Distance to dumpsite.

Given average conditioned for a 46" pipe with a total footage of 10,000 feet, with 18" of silt with 500 to 1200 foot average runs, good accessibility, 500 miles travel one way, less than one hour round trip to the dumpsite: the cost to clean this line would be approximately $6.70 per foot and the time required to complete the project would be 10 - 12 days. This line we are using as an example contains about 1600 cubic yards of debris! That would be 80 loads in a 20 yard sludge trailer.

We need to evaluate each job independently as to the feasibility of using this equipment. We cannot do every job. This unit was designed to move large amounts of solids quickly in a pipeline. If you have a unique cleaning problem, we have the resources to custom design equipment to clean your lines.

There are many new and exciting methods to rehabilitate large diameter sewers. Most have one thing in common... they need a clean line to work in. If you are considering using one of the slip lining methods in your rehabilitation program, this may be of interest. Normally an engineer will specify that the contractor "proof" the line after cleaning and prior to relining. Our cleaning head is actually larger than the new pipe used for lining, therefore cleaning and proofing can be combined into one operation. This can save both time and money for the contractor and the owner.

Those of us contractors who are in the pipeline rehabilitation business find these to be very challenging and exciting times. If contractors, owners and engineers can work together and really listen to each other, we can develop economical solutions for our infrastructure rebuilding program.

BATCH FLUORIDATION OF WATER STORAGE RESERVOIRS

- A MODIFIED PROCESS -

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INTRODUCTION

Dental caries (tooth decay) is widespread in the Northwest Territories (NWT), and its incidence is especially high among native people. It is a multi-factorial disease that is best countered by a combination of good nutrition, improved oral hygiene and the use of fluoridation practices. The revised regulations of the NWT Public Health Ordinance (1980, No. 218) recommend that community water supplies contain between 1.2 and 1.6 mg/L fluoride; fluoridation of water supplies to an optimum concerntration of about 1.4 mg/L is considered to be an effective fluoridation practice, because fluoride strengthens the enamel during tooth formation. The natural fluoride content of fresh water in the NWT is generally quite low (approximately 0.1 mg/L). Therefore, to achieve the optimum fluoride concentration, most communities would have to add fluoride to their water supplies.

In 1986, the Government of the Northwest Territories (GNWT) initiated a health-oriented performance and engineering study on the feasibility of fluoridating community water supplies. The study resulted in the development of the batch fluoridation process as a safe and effective means of water fluoridation in communities that possess year-round water storage reservoirs (Marianayagam, 1986). The process involves the use of a variable flow rate pump to inject hydrofluosilicic acid (fluoride chemical) into the reservoir fill line from a shipping container or feed tank, during the annual reservoir filling operation. The addition of fluoride during the filling of the reservoir was recommended for the following reasons:

a) Thorough mixing of the fluoride chemical with the water would take place due to the turbulence in the supply line between the point of injection and the storage reservoir.

b) Fluoridation of the reservoir would be done once a year and therefore require skilled supervision for a period of time during the reservoir filling process.

A field trial was conducted in 1987 utilizing the reservoir in Tuktoyaktuk, to demonstrate the technology of batch fluoridation (Murray, 1987). The reservoir was successfully fluoridated and has been fluoridated on an annual basis since the field trial.

The process in Tuktoyaktuk involves the injection of the fluoride chemical into the fill line at a concentration that does not exceed 1.6 mg/L; the recommended maximum concentration in potable water according to the NWT Public Health Ordinance. At this concentration, the addition of the fluoride chemical could take approximately 10 to 15 days.

In the Tuktoyaktuk reservoir, the outlet of the fill line is immediately adjacent to the truckfill intake, with both pipes entering the reservoir from the bottom. The close proximity would result in short-circuiting of the fluoride chemical between the two pipe openings. Unless the fluoride is injected at a concentration that does not exceed 1.6 mg/L, short-circuiting would result in unacceptable concentrations of fluoride entering the trucked water distribution system.

[For additional background information on dental health in the NWT and the batch fluoridation program in Tuktoyaktuk, the reader is encouraged to review Marianayagam et al (1988), Murray and Heinke (1988a), Murray and Heinke (1988b) and Johnson and Murray (1990)].

In 1989, the GNWT expanded the batch fluoridation program to include reservoirs in two eastern Arctic communities, viz. Pangnirtung and Igloolik. The fluoridation of these reservoirs was carried out using a major modification of the established process for the Tuktoyaktuk reservoir.

For additional information on the introductory implementation of batch fluoridation in Pangnirtung and Igloolik, including scheduling, procurement and delivery of materials and equipment, dealing with hazardous goods (e.g. fluoride chemical), meetings, liaison and training, the reader is encouraged to review UMA (1990a).

THE MODIFIED PROCESS

The reservoirs in Pangnirtung and Igloolik were fluoridated by injecting the fluoride chemical into the reservoir fill line at concentrations greater than 20 mg/L. The chemical was injected at this rate on a trial basis, in order to try and minimize the amount of time, and subsequent cost, of skilled supervision during the reservoir filling period. The piping configuration in both reservoirs favoured this approach, because unlike the Tuktoyaktuk reservoir, the distances between the fill line outlet and truckfill intake are relatively large, particularly in the Pangnirtung reservoir.

The fluoridation systems utilized in Pangnirtung and Igloolik are illustrated in Figures 1 and 2 respectively.

PRECAUTIONS

Several precautions were taken to ensure that the modified process remained a safe and effective means of adding the fluoride chemical to year-round storage reservoirs. These are outlined below:

i) Water samples were collected from the reservoir and analyzed for fluoride concentrations at least once each day. The location of water sampling points was established by an approximate grid across the reservoir. Figures 3 and 4 indicate the approximate water sampling locations for Pangnirtung and Igloolik respectively.

   Water samples were collected at various depths at these locations to monitor the dispersion of the fluoride chemical plume through the reservoir. The results of these samples are presented in Table 1 for Pangnirtung and Table 2 for Igloolik. The results indicated there was a relatively uniform and rapid dispersion of the chemical plume between the outlet of the reservoir fill line and truckfill intake. There was no evidence of short-circuiting of the fluoride chemical between these two locations, which would have resulted in the withdrawal of water with excessive concentrations of fluoride.

   The collection and analysis of water samples was also an excellent means of checking fluoride concentration calculations. It was important to ensure that the initial fluoride concentration of each reservoir was as close as possible to the desired concentration (refer to discussion below on "Freeze Concentration of Fluoride").

ii) In addition to the sampling/testing program, samples were occasionally taken from the truckfill arm during the filling of water trucks to check whether the fluoride concentrations were approximately equal to the fluoride concentrations measured in samples collected from sampling points in the reservoir, close to the truckfill intake. In all such tests, the differences were insignificant.

FREEZE CONCENTRATION OF FLUORIDE

It has been established that impurities including most salts and ions such as fluoride, are rejected from freezing water making the ice relatively pure (Fertuck, 1968 and Graydon, 1986). The more slowly ice is formed, the more efficient this process becomes. The net result of this "freeze concentration" phenomenon is that impurities become concentrated in the water remaining below the ice.

Fertuck (1969) reports that the upper layers of ice are generally much purer than the rest of the ice and attributes this to the fact that the upper ice is made up of crystals which have a vertical c-axis (the c-axis or crystallographic axis is the axis around which the ice crystal develops). The lower layers of ice are made up of crystals that have a horizontal c-axis. According to Fertuck (1969), ice crystals with a horizontal c-axis contain much more impurities than ice crystals with a vertical c-axis.

The exclusion of fluoride from the ice phase is dependent to a large extent on the change in freezing point due to the dissolution of the fluoride ion (Graydon, 1986). It is not expected that a fluoride ion concentration of 1.4 mg/L will alter significantly either the rate of freezing or the freezing point of the ice. Graydon (1986) reports that there is no known technique for calculating the amount of fluoride excluded from the ice into the water beneath.

Based on the above, it has been conservatively assumed in all NWT batch fluoridation practices to date, that 100% of the fluoride in the ice is rejected from the freezing water. Therefore, one must ensure that the initial (full reservoir) fluoride concentration does not produce a concentration in the unfrozen portion of the reservoir, above the recommended maximum of 1.6 mg/L at the maximum ice thickness.

The reader is encouraged to review UMA (1990b and 1990c) for calculations to establish the desired initial fluoride concentration in the Pangnirtung and Igloolik reservoirs respectively.

DISCUSSION OF FINDINGS

The modified process was a success. It not only achieved the required end results, but enabled a significant reduction in duration, and subsequent cost, of skilled supervision of the fluoridation process during the reservoir filling period. The amount of time spent in each community was reduced to about 4 to 5 days.

The modified process can be a safe and effective means of adding the fluoride chemical to year-round storage reservoirs, as long as the necessary precautionary measures are observed. Further, it is evident that the process will probably succeed in the Tuktoyaktuk reservoir too, if arrangements can be made to suspend the trucked delivery system for the duration of fluoridation. In this way, there can be no short-circuiting of fluoride chemical in the reservoir.

While this paper focuses on a modified approach to the batch fluoridation of year-round water storage reservoirs in the NWT, there is no reason why the technology cannot also be utilized in southern (Provincial) communities with similar water supply systems. In this way, batch fluoridation will extend the dental health benefits of water fluoridation to those communities which do not have conventional water treatment plants. Where a conventional water treatment plant exists, the following can be said: The technology of water fluoridation has been established for decades, and all that is required is for these communities to obtain community approval and to provide the facilities for fluoridation.

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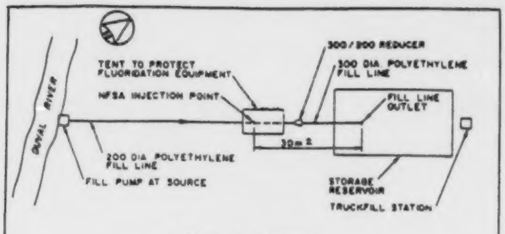
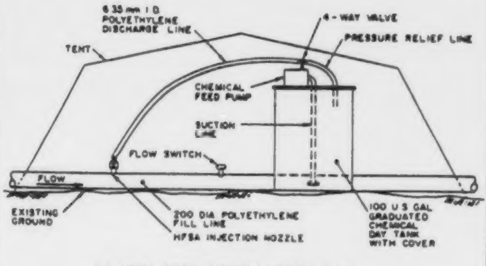


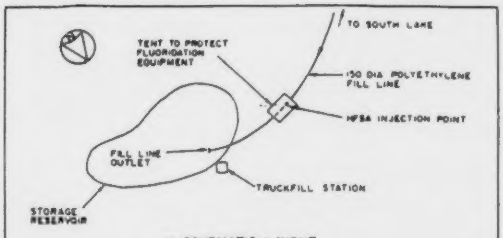
a.) SCHEMATIC LAYOUT
N.T.S.


6.35 mm I.D. POLYETHYLENE DISCHARGE LINE
4-WAY VALVE
PRESSURE RELIEF LINE
TENT
CHEMICAL FEED PUMP
SUCTION LINE
FLOW SWITCH
FLOW
EXISTING GROUND
200 DIA POLYETHYLENE FILL LINE
HFSA INJECTION NOZZLE
100 U.S. GAL GRADUATED CHEMICAL DAY TANK WITH COVER

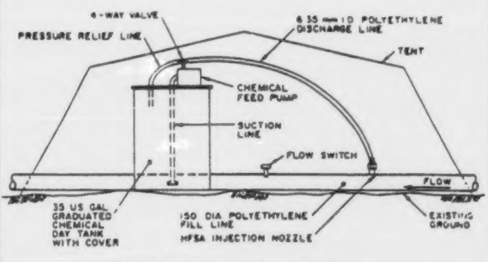
b.) HFSA FEED SYSTEM INSIDE TENT
N.T.S.

PANGNIRTUNG
FLUORIDATION SYSTEM

FIGURE 1



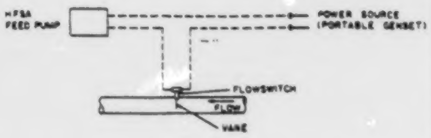
a.) SCHEMATIC LAYOUT
N.T.S.


4-WAY VALVE
PRESSURE RELIEF LINE
6.35 mm I.D. POLYETHYLENE DISCHARGE LINE
TENT
CHEMICAL FEED PUMP
SUCTION LINE
FLOW SWITCH
FLOW
35 US GAL GRADUATED CHEMICAL DAY TANK WITH COVER
150 DIA POLYETHYLENE FILL LINE
HFSA INJECTION NOZZLE
EXISTING GROUND

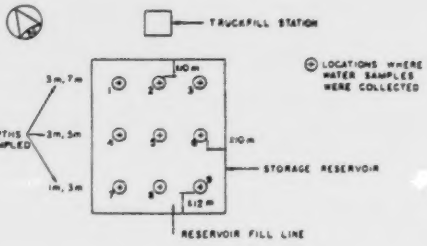
b.) HFSA FEED SYSTEM INSIDE TENT
N.T.S.

IGLOOLIK
FLUORIDATION SYSTEM

FIGURE 2



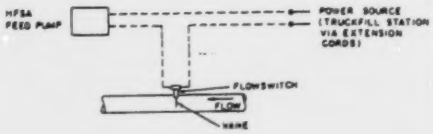
ELECTRICAL CONNECTION - SCHEMATIC
N.T.S



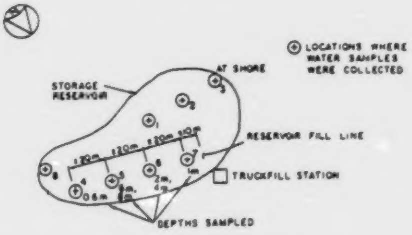
WATER SAMPLING LOCATIONS
N.T.S

PANGNIRTUNG

FIGURE 3



ELECTRICAL CONNECTION - SCHEMATIC
N.T.S



WATER SAMPLING LOCATIONS
N.T.S.

IGLOOLIK

FIGURE 4

TABLE 1

FLUORIDE MONITORING DURING 1989
ADDITION OF FLUORIDE - PANGNIRTUNG RESERVOIR
FLUORIDE CONCENTRATION AT VARIOUS LOCATIONS, DEPTHS AND TIMES

DATE DAY/MON/YR	TIME OF SAMPLING	SAMPLE TAKEN AT DEPTH (m)	LOCATION NUMBER								
			1	2	3	4	5	6	7	8	9
02 09 89	15:30	N/A	-	-	-	-	-	-	-	-	-
	16.00	0	0.1	0.1	-	-	0.1	-	-	-	-
03 09 89	09:00	0.0	-	0.25	0.2	-	0.3	0.3	-	0.25	0.1
		1.0	0.2	-	-	-	-	-	-	-	-
		2.0	-	-	-	0.3	-	-	-	-	-
		4.0	-	-	-	-	-	-	0.3	-	-
04 09 89	09:15	0.0	0.62	0.61	-	0.50	0.45	-	0.42	0.40	-
		1.0	-	-	0.58	-	-	-	-	-	-
		3.0	-	-	-	-	-	0.45	-	-	-
		5.0	-	-	-	-	-	-	-	-	0.42
06 09 89	09:15	1.0	0.83	0.81	0.81	-	-	-	-	-	-
		2.0	-	-	-	0.72	0.73	0.75	-	-	-
		3.0	0.82	0.81	0.79	-	-	-	0.78	0.75	0.76
		5.0	-	-	-	0.75	0.72	0.78	-	-	-
		7.0	-	-	-	-	-	-	0.79	0.76	0.75

NOTE: - Sample locations are shown in Figure 3

TABLE 2

FLUORIDE MONITORING DURING 1989
ADDITION OF FLUORIDE - IGLOOLIK RESERVOIR
FLUORIDE CONCENTRATION AT VARIOUS LOCATIONS, DEPTHS AND TIMES

DATE DAY/MON/YR	TIME OF SAMPLING	SAMPLE TAKEN AT DEPTH (m)	LOCATION NUMBER							
			1	2	3	4	5	6	7	8
10 09 89	15:00	0	0.5	0.5	-	-	-	-	-	-
		1	-	-	-	-	-	-	0.60	-
		4	-	-	-	-	-	0.54	-	-
		8	-	-	-	-	0.60	-	-	-
11 09 89	11:20	0	-	0.70	0.60	0.59	-	-	-	0.80
		1	-	-	-	-	-	-	0.60	-
		4	-	-	-	-	-	0.59	-	-
		8	-	-	-	-	0.60	-	-	-
11 09 89	20:20	0	-	0.71	-	0.71	-	-	-	0.71
13 09 89	09:45	0.0	-	-	0.92	-	-	-	-	0.85
		0.3	0.85	0.85	-	-	-	-	-	-
		0.6	-	-	-	0.80	-	-	-	-
		1.0	-	-	-	-	-	-	0.80	-
		2.0	-	-	-	-	-	0.81	-	-
		6.0	-	-	-	-	0.78	-	-	-

NOTE: - Sample locations are shown in Figure 4

THE BENEFITS OF SELF PRIMING SEWAGE PUMPS IN LIFT STATIONS

F. RUTLEDGE, GORMAN-RUPP OF CANADA LTD.

INTRODUCTION

Sewage lift stations are frequently designed utilizing conventional wet well designs and equipment specifications that are well understood and readily available from past projects.

Unfortunately, these designs often require that the pumps be located in the wet well itself or in an adjacent dry well. Both of these arrangements are subject to safety hazards and inconvenient accessibility for maintenance.

With the emphasis today on "worker safety" in the work place and on equipment reliability suitable for continuous, unattended operation, alternative wet well and pump designs that minimizes the workers' exposure and provides system reliability should be considered.

While the "self-priming" lift station design is not new (self-priming pumps for sewage applications have been available for over 30 years), it is offered in this paper as the available alternative that meets the criteria of safety and reliability.

DESIRABLE FEATURES OF A SEWAGE PUMP

Pump design

- Non clog design impeller with 2.5 " or 3 " solids handling capability and pump out vanes on the back to keep solids away from the seal and minimize seal pressure.
- Ductile iron impeller which has better wear resistance than cast iron.
- All internal liquid passages capable of passing the published solids handling size of the pump.
- Large and easily accessible clean-out to provide complete access to the impeller, seal and wear plate for easy inspection, cleaning and replacement if necessary.
- Easily removable rotating element and other parts subject to wear for service without disturbing the suction or discharge piping.
- Externally adjustable wear parts.
- Mechanical seal and replaceable shaft sleeve with a clean, self contained lubricating environment (not dependant on lubrication from the liquid being pumped) and specifically designed for sewage applications. Materials should be stainless steel metal parts with viton elastomers for solvent resistance and tungsten titanium rotating and stationary faces (for long life against abrasion) with a self aligning feature to compensate for shaft deflection.
- Heavy duty construction with bearing housing cooling.
- Relatively efficient.
- Capability of using V-belt drives for design flexibility.
- Utilizing a proven overall design with a minimum of 5 years experience with sewage applications.

Drive options

- The pump arrangement should be suitable for use with any user preferred type of standard drive and motor that is readily available.

Serviceability (pump and drive)

- Installation out of the wet well to provide easy serviceability and preventive maintenance checks.
- simplistic in design to allow for on site or local repairs when required and without special tools or equipment.
- Good availability of parts, service and technical assistance as required.

THE SELF-PRIMING PUMP - HOW IT WORKS

The basic components of a self priming sewage pump are illustrated in Figure 1 and generally consists of:

- initial priming port
- water / air separation chamber
- repriming port
- air release valve
- clean-out cover
- non clog impeller
- inlet check valve
- system check valve (part of the piping supply)
- volute drain

A self-priming pump requires that water be added to the volute casing only at the time of initial start-up. On subsequent starts, the pump will be full of liquid and will respond immediately provided that the system and/or the pump suction check valves are functional. Should both these valves be temporarily blocked open when the pump shuts down, the pump volute will still be about half full of liquid which is sufficient for the pump to reprime. The manufactures' published data should be based on this possibility.

The function of a self-priming pump is to effectively evacuate the suction line and pump volute of air and begin to pump at the specified performance within a reasonable time.

Upon startup, the liquid/air mixture is pumped into the self contained volute reservoir where the air and liquid separate. The air rises and the liquid drops down and is re-introduced to the impeller through the repriming port. This action creates a low pressure area at the eye of the impeller which draws in more air from the suction line which is mixed with the recycled liquid and this cycle repeats until the pump is fully primed at which time it operates and is controlled in the same manner as a standard centrifugal pump.

As the pump becomes primed, the air collects at the top of the volute and is forced out the discharge. Most systems will require an air release valve to effectively exhaust the air back to the wet well. This valve should be specifically designed for solids applications and should shut off when the pump is primed to avoid inefficient continuous bypassing.

SUBMERSIBLE PUMP LIFT STATIONS (FIGURE 3)

Advantages

- The equipment available offers good hydraulic coverage.
- The wet well is often a simple inexpensive design which results in a lower initial installed cost than other alternatives.
- There is a limited amount of above ground structure required.
- The running equipment is submerged in the wet well, therefore, noise is not a factor.
- There is no drive train between the motor and the pump to maintain.
- Accidental flooding of the station is not a concern.

Considerations and limitations

- All mechanical equipment including the electric motor is submerged in the wet pit and as a result:
  - The pumps and motors are not readily accessible for maintenance or routine inspections for bearing noise, cavitation noise or seal leaks.
  - An electrician may be required to disconnect the pump/motor for service.
  - Lifting devices are required to remove the equipment for service.
  - Entry into the wet well may be necessary to clear jammed equipment.
  - Trouble shooting the pump performance is difficult with no way of confirming the suction condition with a gauge reading.
- Seal failure detection and/or shutdown devices often produce nuisance tripping and are frequently disconnected negating the feature.
- Because the condition of the seals and bearings is hard to monitor, the equipment is often run until a complete failure occurs, usually catastrophic.
- The submersible motor is a specialized piece of equipment which requires trained personnel, both mechanical and electrical, for service and often must be returned to a factory service centre for repairs.
- Repair parts and service are normally very expensive with long lead times.
- Equipment must be removed from the wet well to adjust the impeller clearances.
- The guide rails and quick disconnect elbow must be maintained by entry into the wet well.
- The connection between the base elbow and the pump flange can leakage with a loss in efficiency and can be difficult to detect and expensive to correct.
- With no speed change possible on fixed speed units, a requirement to change the hydraulic conditions can be difficult at best and expensive once the pumps are selected and installed.
- Auxiliary standby power must be provided through a generator set.
- The potentially hazardous environment in the wet well must be considered more than with any other lift station design.
- The minimum submergence for motor cooling may limit the wet well low level.

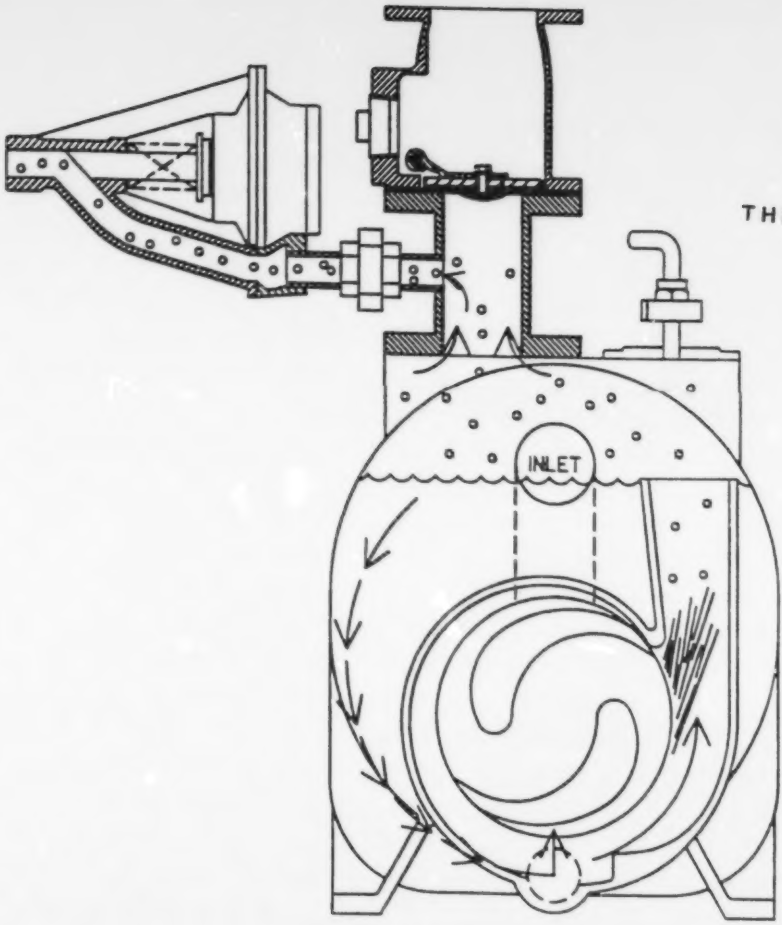


FIGURE 1

THE SELF-PRIMING PUMP

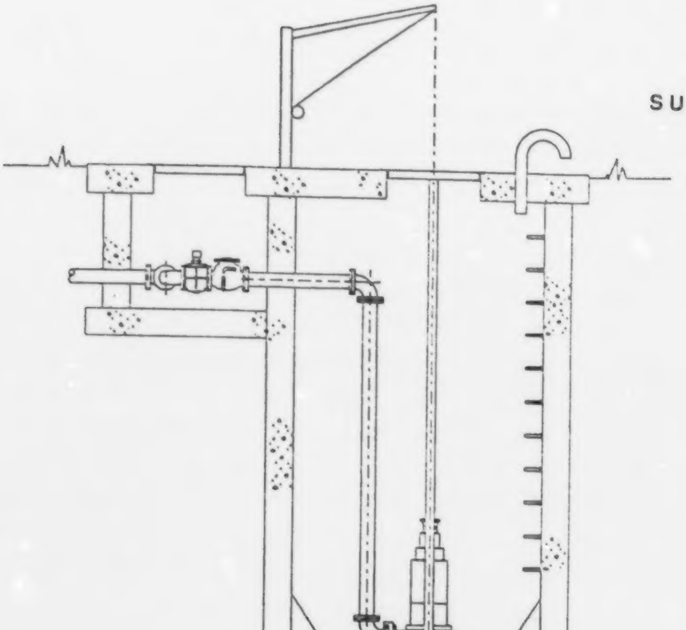


FIGURE 3

SUBMERSIBLE LIFT STATION

WET WELL / DRY WELL PUMP LIFT STATIONS (FIGURE 4)

This basic lift station design can be one of several modified versions but usually includes the use of one of the following types of dry pit, end suction, standard centrifugal sewage pumps:

- horizontal with direct or belt drive.
- vertical mount on a base elbow with a close coupled direct drive.
- vertical mount on a base elbow with an extended shaft and a direct drive motor mounted on a floor above motor.

Advantages

- All mechanical equipment is out of the wet well and generally accessible for maintenance and routine maintenance.
- The equipment available provides a good hydraulic range.
- Suction and discharge gauge readings can be easily obtained for trouble shooting the pump performance.
- Generally, depending on the actual equipment design, all major repairs can be done by regular maintenance personnel.
- In some designs, the use of standard motors is possible.

Considerations and limitations.

- An elaborate and expensive structure is required.
- Lifting devices are required if the equipment has to be removed for service.
- Depending on the pump design, adjusting impeller clearances can be difficult or impossible.
- The packing or mechanical seal may require a water flushing system.
- The dry well is subject to accidental flooding especially with a packed pump seal.
- A sump pump is required to handle any leakages from the pump or seal flushing arrangement.
- The clean-out opening is usually too small or non existent.
- Additional valving is required between the wet well and the dry well.
- The dry well requires heating and ventilating.
- The pumps have limited air handling capability with no automatic air release feature.
- If the pumps are direct driven, it will be difficult to change the hydraulic performance if required.
- By design, the dry well must be as deep as the wet well requiring long stairways or even elevators.
- The potential hazards of wet well gases must be constantly considered.

SELF-PRIMING LIFT STATIONS (FIGURE 2)

Advantages

- All mechanical equipment is high and dry out of the wet well or adjacent dry well and as a result:
  - pumps and valves are accessible for maintenance and routine inspections in a dry, non-hazardous environment
  - no lifting devices needed for most maintenance procedures.
  - preventive maintenance checks, for such items as oil levels, bearing noise, seal leaks and even cavitation, are easy and more likely to be followed without the requirement to remove the equipment or to enter the wet well.
  - pump troubleshooting is simplified with the capability of using suction and discharge gauges.
- All major repairs can be performed on site by regular maintenance personnel and no special tools or training is required.
- The station is generally located above the wet well and is not subject to flooding.
- If an air bubbler system is used for level control, there is no electrical equipment in the wet well and therefore wet well ventilation is minimal.
- The occasional clog can be easily cleared in minutes through the access provided by the large clean-out cover.
- The pumps can be drained by gravity back to the wet well which minimizes any mess and the workers exposure to the sewage.
- It is quick and easy on a well designed pump to reset impeller-to-wear plate clearances to maintain 100% of the original pump efficiency.
- It provides inexpensive maintenance due to the use of off-the-shelf standard horizontal foot mounted motors (high efficiency motors readily available if specified) and drives.
- It is simple and inexpensive to alter the hydraulic conditions with the use of V-belt drives.
- Complete package stations, above and below ground types, can be provided by some manufactures which includes pumps, valves, piping, controls and a fiberglass enclosure all factory assembled and tested prior to shipment providing single source responsibility.
- Auxiliary standby power can be used to drive the pumps without the use of a generator set.

Considerations and limitations

- Buildings generally will require heating and ventilation.
- The static lift must be within the capability of the selected pump. Depending on the actual pump model, speed and suction piping arrangement involved, lifts up to 25 feet can be handled with self-priming pumps with 15 feet being a typical ideal design lift.

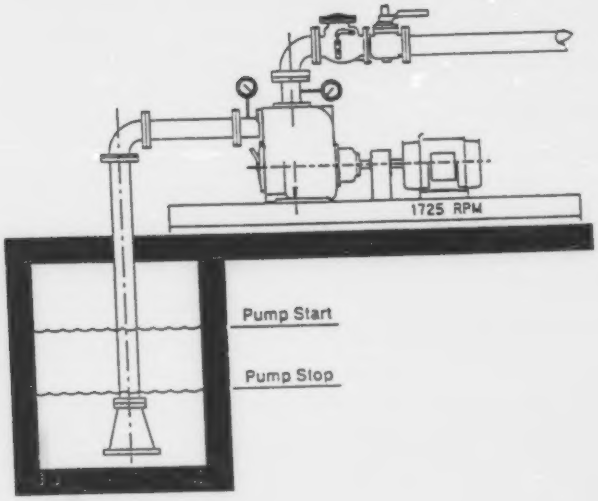


FIGURE 2

SELF-PRIMING LIFT STATION

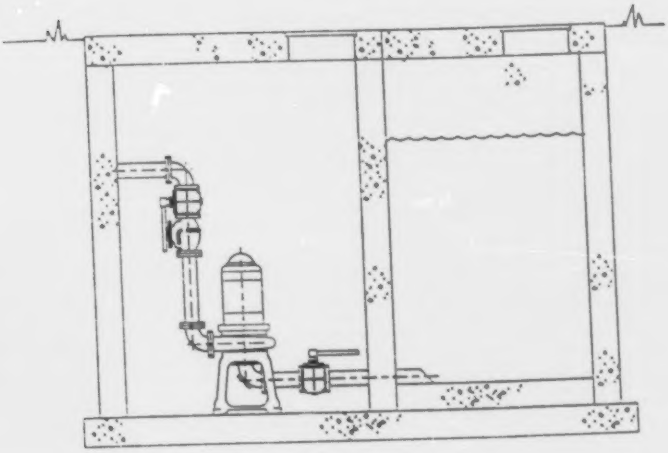


FIGURE 4

WET WELL/DRY WELL LIFT STATION

HDPE REHABILITATION
OF CURVED SEWER LINES
WITHOUT FLOW INTERRUPTION

Lee N. Sparks - Plexco/Spirolite*
Harold R. Smith - Plexco/Spriolite**
Al A. Heinisch - Plexco/Spriolite ***

INTRODUCTION

In the past several years, sanitation districts across the nation are finding that a large portion of their key sewer lines are in need of replacement. The interior of these lines has corroded over time due to the emission of sulfide gases from the sewage. Left unattended, the pipeline can eventually deteriorate to the point of losing structural integrity. A proven method for rehabilitating the damaged pipe is to line it with a smaller diameter pipe made of a material that is resistant to the chemicals in the sewage. A material frequently used in this method is high density polyethylene (HDPE).

HDPE pipe in twenty feet lengths can quickly line the straight portions but what about the curves in the sewer line? In the past, these curved sections were bypassed entirely or were excavated and a liner pipe was lowered in. If excavation was not possible, the curved section could be repaired by patching the damaged portions or by lining it with a cured-in-place pipe. In both cases, the sewage flow must be stopped while repairs are made. This can be rather inconvenient if the section in case is a major artery of the system. A definite need for rehabilitating the curved sections without interrupting the flow was present.

A method to meet this need has been in use in several projects for the past few years. The INTEGRA-SLINKS process, patented by the Spirolite Corporation, utilizes a series of short lengths of HDPE pipe with specially designed, gasketed bell and spigot joints. As the short lengths, slinks, are inserted into a curve, the joints open or articulate to allow the liner's forward motion to follow the direction of the curve. The articulation of the joints is controlled by a specially designed system. Large diameter curved pipe lines with radii of curvature smaller than 20 feet may be rehabilitated.

ADVANTAGES

As stated by the title, a main advantage of using slinks is the ability to rehabilitate a curved section of a sanitary sewer without interrupting flow. This is important especially in large diameter trunk sewers where flow termination or bypass pumping is difficult, if not impossible.

The use of slinks eliminates the need for a long, open trench. The only digging required is that for an insertion pit. If pit locations are planned well, an insertion pit can be used to install both slinks and standard liner pipe in one direction or the other. This factor is particularly useful for projects where the damaged pipe is at a depth greater than 15 feet.

Installation of the slink modules can be done quickly, depending on the length and condition of the curve. A swift completion of the pushing procedure allows the pit to be open for a minimum amount of time. Environmental concerns are negated by the short duration of the open pit.

Slinks can rehabilitate curved sewer lines beneath roads, buildings, and other permanent structures without surface damage. The expense of restoring these structures to their original condition, after repairing the damaged pipe, is removed.

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DESIGN

A. Curve Modeling

The dynamics of liner insertion in these sharp curved cases involve not only the requirement to achieve layout to a specific radius of curvature, but also the need to negotiate tight, restrictive annular spaces without becoming bound. All curves within the existing system are modeled and trail liner insertions on a computer equipped with CAD software are performed. The results of these computer trials form the basis of the specialized liner design as well as evaluate the success of the proposed insertion. Consequently, accurate modeling of the existing curve with the best available data is of prime importance to project success.

On most project plans, the curve data supplied is fairly standard: Delta, the overall angle of the curve; R, the curve radius; L, the length of the curve; and T, the length of a line tangent to a curve end. This information is then used to construct the arc that the pipe curve will follow. See Fig. 1.

In actuality, the pipe "curve" is not a true curve, but is instead, a number of straight pipe lain with angles at the connecting joints. The sum of these angles add up to form the angle, Delta. The angles, Delta/N, can be formed either by pulling the joints open or by using radius pipe which has mitered joints. For slink design purposes, an assumption is made that the pipe found in the curve is radius pipe. This provides a more conservative model by reason of the shorter lengths involved with radius pipe.

Using accepted design procedures (1), the angles at the joints, Delta/N, can be determined using the following equation and givens:

$$\text{Tan } \Delta/N = L/(R + D/2 + t)$$

N = Number of radius pipe
L = Standard laying length of pipe being used, feet
R = Radius of curvature, feet
D = Inside diameter of the pipe, feet
t = Wall thickness of the pipe, feet
Δ = Delta, total deflection angle of curve, degrees
Δ/N = Total deflection angle of each pipe, degrees

Note that the Delta/N angle calculated by this formula is an average over all the mitered pieces. From the information determined, a model can be created in the manner o the original design. See Fig. 2.

However, in the actual construction of the pipe curve, it is unlikely that each angle found at the joints will be equal to the Delta/N angle calculated above. The sum of the miter angles found could add up to the overall Delta angle and yet still differ considerably from the calculated miter angle. Also, the lengths and number of the pipe sections could differ from the designed lengths. For these reasons, to build a true model of the curve, it is highly recommended that a field survey be performed to determine the actual Delta angle, the miter angles of the pipe sections, and the number and laying lengths of the pipe forming the curve. Any misaligned or kicked joints in the curve should also be noted as sources of obstruction. Obviously, the more accurate the information on the actual curve is, the better the model and the rehabilitation design will be.

B. Initial Liner Calculations

Once the design curve model is developed, a basic design of the slink liner pipe is needed. Design of these special joints encompasses evaluation of burial performance as well as insertion considerations. The considerations of most importance to burial are, as follows:

1) depth of cover above pipe crown
2) height of groundwater surface above pipe
3) extent of live loads

From these factors, a wall thickness possessing a sufficient moment of inertia, centroid, and average profile area to ensure adequate performance can be calculated. Typical calculations performed to determine the liner pipe's effectiveness in the given conditions include: buckling resistance; wall crush strength; pipe deflection; and pipe bending strain. Any values specified by the customer for use in these calculations should be noted.

Once the necessary wall thickness has been determined, the design for the slink liner can be further defined. A typical slink design will have a laying length of approximately eighteen inches and an allowable articulation of 3 to 3.5 degrees, depending on the diameter and joint design. Before proceeding, the outer diameter of the liner pipe and the inner diameter of the pipe being repaired are compared to determine the annular space remaining or an interference between the two pipes. After completing the slink design, an insertion test can be conducted.

C. CAD Insertion Test

Once the initial design is set, a scale model of the slink, complete with a bell and spigot joint, is constructed using the CAD software. See Fig. 3. Several slink models are connected together at the joints, forming a train. The train of slinks is then positioned at the entrance of the curve model previously competed, parallel to the first pipe section in the curve. The train is then moved forward into the curve until the lead slink comes in contact with the wall of a curve pipe section. The contact occurs at the mitered joint between the first two pipe sections in the curve. At that time, the joint between the first and second slinks opens up, or articulates, moving the lead slink out of contact with the curve wall. The forward push is then resumed until the train once again comes into contact with the curve wall. If the first joint in the train has already reached the allowable articulation limit, the next articulation will occur at the second joint, once again moving the slink train out of contact with the curve wall. This procedure continues until either the curve has been fully lined or the train is wedged inside the curve and can proceed no further.

The most important point of observation during an insertion test is the action of the slink train as it passes through the individual mitered joints of the curve. This action is directly related to three areas of the slink's design: the outer diameter, the overall length, and the allowable joint articulation. These factors govern the slink's ability to move through the various angle points of a curve. In curves having miter angles smaller than the slink's allowable articulation, the liner joints simply open to that angle as they pass by. However, if the miter angles are larger than the slink's capacity, the liner train must span each angle over at least two slink's joints.

The latter case is seen in the majority of projects using this lining method and is where the outer diameter and overall length are most important. The slink's outer diameter and overall length are crucial in spanning sizeable miter joints for one reason: space. The outer diameter determines how much annular space remains between the liner pipe and the damaged pipe. The overall length is a key factor in the volume of space taken up by the articulation of two or more joints.

If the slink train becomes wedged in a curve, the probable cause involves a combination of the three areas described above. The curve miter angles are usually of a size that requires two or even three slinks joints to articulate to form an equivalent. At the start of the insertion, the train may have had enough annular space to pass through the first two miter joints in the curve. As the slinks continue, the curve direction forces the train to articulate joints farther down to accommodate the progressively greater sum of the angles. The number of joints opening requires a larger volume of space as the liner proceeds. Eventually, the space runs out and the liner pipe becomes wedged.

If the insertion test fails in this manner, the slink design must be modified. The first two points of alteration would be to shorten the laying length of the liner and increase its allowable articulation. If the altered design is still unable to pass through the curve, the wall height can be re-evaluated to determine if a decrease is possible. At this point, if liner pipe is still incapable of going through the entire curve, the only recourse is to use a pipe with a smaller inner diameter. The use of this choice requires a comparison of the flow volume for the smaller pipe with the project's flow requirements. Once the design is finalized, manufacture of the liner pipe can begin.

MANUFACTURE

The manufacture of a slink begins with a short length of a solid wall high density polyethylene pipe. The designed features of the slink are machined out of the pipe by a vertical lathe controlled by computerized numerical control (CNC). The use of the computer in directing the lathe's actions allows for close tolerances for the dimensions to be set and held. The lathe is capable of constructing slinks of inner diameters from 18 to 72 inches in various laying lengths.

After the slinks are produced, an external cable system which controls the articulation of the joints is installed. Cable placement and length are closely coordinated to set allowable joint articulation within the train. This placement incorporates both the viscoelastic properties of the materials and the dynamics of insertion such as train rifling.

The cabled liner pipe joints are assembled into modules of usually five slinks to ease handling. The slinks are connected spigot-to-bell after seating the gaskets in the spigot grooves. When the pipes are joined, the free ends of the cables within the module are connected to the preceding slink at the groove prepared. See Fig. 4. On ends of the first and last slinks in the module, cables are placed in different locations than the others. See Fig. 5. This placement allows better access to the cables when connecting one module to another in the insertion pit. The assembled modules are then ready for shipment to the project site.

INSTALLATION

The curve to be rehabilitated must be prepared before beginning the liner insertion. Any obstructions observed in the curve, such as misaligned joints, and the direction of the flow should be taken into consideration when choosing an insertion pit location. Prior to insertion, the pipe curve must be cleaned to remove articles such as sand, gravel, trash, etc. A prover piece of the same outer diameter as the slinks needs to be pulled through the curve after cleaning to detect any remaining sources of obstruction. Upon satisfactory completion, the curve is ready for the liner installation.

Installation of the slink liner pipe requires the use of certain equipment. The machine performing the installation should be equipped with a full-face pushing head. This structure will distribute the jacking force around the perimeter of the pipe. The pushing mechanism should also be rigged with a device to measure the jacking force being applied. The jacking force should not exceed the determined allowable limit.

Upon insertion, it is recommended that the slink train be pushed into the curve with the spigot end first to ease the installation, but this does not preclude bell first insertions. The placement of a nosecone at the train's leading edge is also advised to facilitate directional changes and protect the front spigot.

GROUTING

Grout will be pumped into the remaining annular space after the liner insertion is completed. When grout is pumped in place, the liner pipe should be surcharged with sewage or water to aid in the control of flotation. The primary function of the grout is to provide support for the flexible liner pipe. An analysis of the liner's designed properties determines the maximum allowable pressure and number of lifts for the injection of the flowable grout. This pressure and certain details of the pipeline are critical in outlining a proper grouting procedure.

An important detail to consider in the section to be grouted is the slope of the pipeline. The increased head pressure from the elevation change in the line can raise the pressure seen at the lower end past the allowable limit. This difference must be considered when setting the grout injection rate.

The set-up for the grouting begins with the installation of bulkheads to restrain the grout. The bulkheads must be equipped with vents. The venting capacity provided should be sized to the projected grout injection rate to prevent pressure build-up once the annular space is liquid filled. Water or sewage in the annular space after the liner installation is removed through the vents or lower side drains (if provided) as it is displaced by the grout.

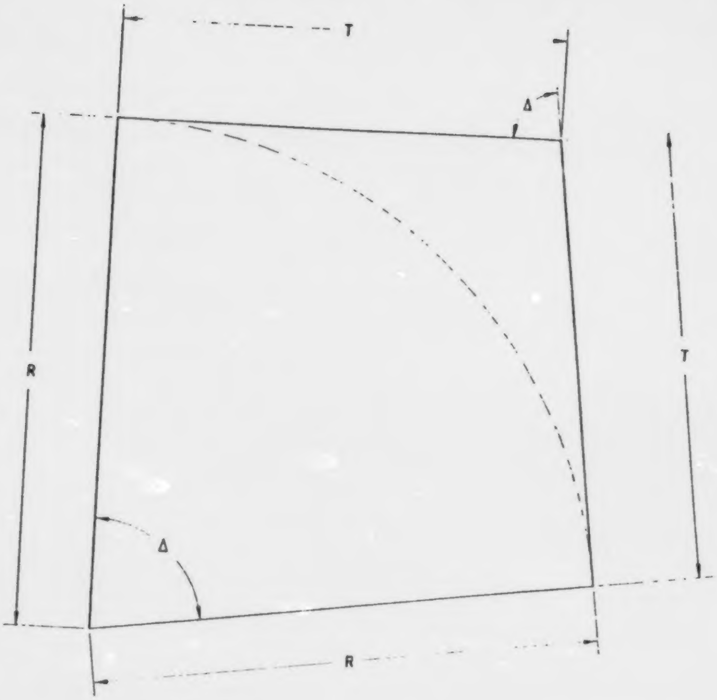
The grout used should be one with a low viscosity and an adequate compressive strength. Sand-based grouts do not have a high flowability and should not be used.

ACKNOWLEDGEMENT

Larry J. Petroff of PLEXCO/Spirolite for his contribution in providing information of this product and advising in its use.

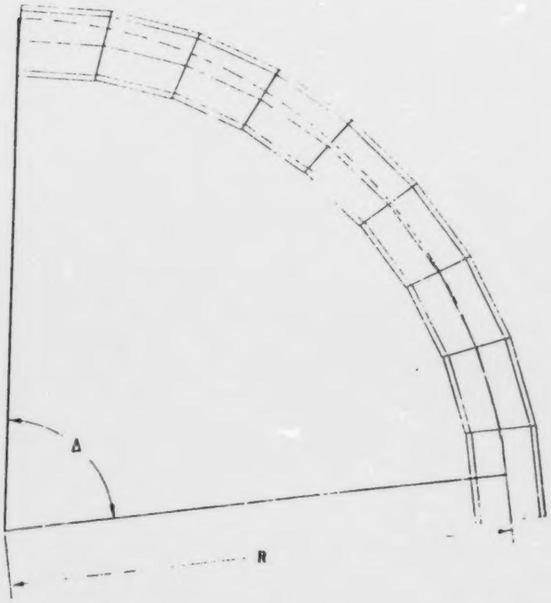
REFERENCES

(1) American Concrete Pipe Association; Concrete Pipe Handbook; 1980; pp. 5-17/22.



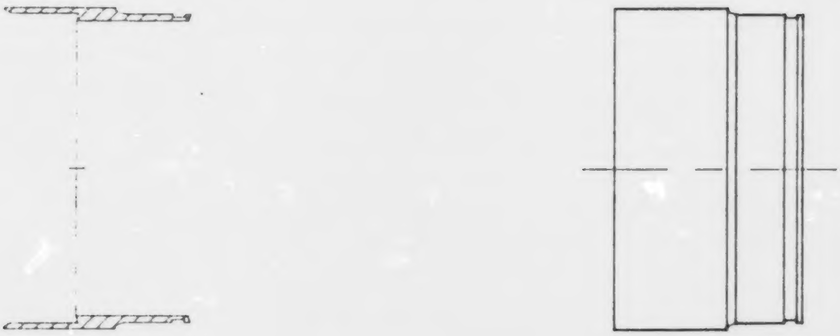
CURVE DATA	
R	45.00'
Δ	82° 20'05"
T	39.35'
L	64.66'

FIG. 1: ARC DESIGN FOR CURVE CONSTRUCTION



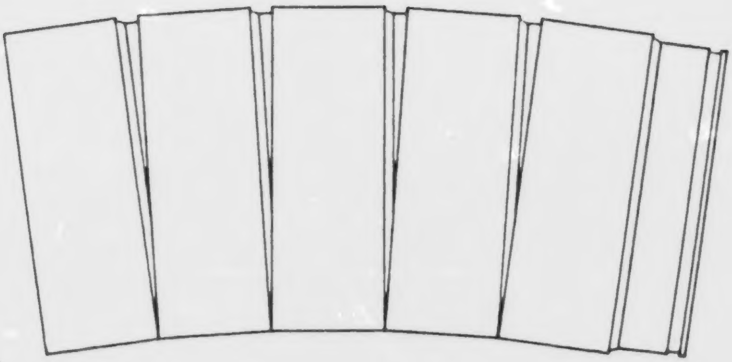
CURVE DATA	
R	45.00'
Δ	82° 20'05"
T	39.35'
L	64.66'
ID	63"
N	8
LL	8'
Δ/N	10°'

FIG. 2: PIPE CURVE FOR GIVEN DATA



CROSS-SECTION

WHOLE



ARTICULATED MODULE

FIG. 3: SLINK MODELS

PRODUCTIVITY MEASUREMENT: A CASE STUDY OF THE CITY OF WINNIPEG'S WATERWORKS MAINTENANCE CREWS

Stanley Siu, P. Eng.
The City of Winnipeg

INTRODUCTION

The City of Winnipeg is one of the few Canadian municipalities who utilize the service of industrial engineers in public works to improve work methods which can lead to increased productivity and level of service.

In 1987, the Industrial Engineer in the Operations Department was assigned to work in the Water Sub-section of the Department, with the objective to improve the work methods and productivity of the 11 waterworks maintenance crews. The project began with a series of work studies conducted in various Districts, including field observations to get to know the operations on a first-hand basis. Based on the results of the analysis, the Industrial Engineer developed and implemented an easy-to-use method to monitor the productivity of waterworks maintenance crews. The productivity monitoring process has resulted in continued productivity improvement since its implementation in 1987. The purpose of this paper is to describe how the productivity measurement method was developed and the impact it made on improving productivity within a relatively short timeframe.

BACKGROUND

The Operations Department of the City of Winnipeg is responsible for the delivery of various municipal services, including public works and in-house construction programs, the control of new subdivision development, supplying and maintaining an equipment fleet utilized by several City departments, and providing primary materials and stores services.

To facilitate the effective delivery of municipal services to the 625,000 citizens of the City of Winnipeg, the city is geographically divided into six Districts: 1 to 6. Each District is responsible for providing streets maintenance, refuse, waterworks and sewer distribution services, as well as technical and administrative services and the control of new subdivision development in its own designated area. The boundaries of the six Districts are shown in Appendix "A".

WATERWORKS MAINTENANCE CREWS

Each of the 11 waterworks maintenance crews within the Operations Department is responsible for the repair and maintenance of the water distribution system within their own District, including the maintenance of watermains, valves, hydrants, and services. Please refer to **Table 1** for the structure of the waterworks maintenance crews.

TABLE 1

STRUCTURE OF WATERWORKS MAINTENANCE CREWS

District	No. of Watermain Crews	Crew Structure
1	2	1 Senior Pipelayer
2	1	1 Junior Pipelayer
3	1	1 Skilled Labourer
4	2	1 Backhoe Operator
5	2	
6	2	

District 3 operates with 1 watermain crew and 1 service crew, making a total of two crews. The service crew, operating with 3 workers (2 Skilled Labourers and 1 Backhoe Operator), is responsible for attending service repairs only. All the crews are assisted by support staff in their own districts, including staff who provide the material logistics to the job sites, watermain turn-on/-off service by the Water Supply Workers, and dump trucks used for excavation purposes. All the watermain crews share the same crew structure, regardless of which District they are stationed in.

WATER MAINTENANCE WORKLOAD DISTRIBUTION

There are a total of 2,338 kilometres of watermains in Winnipeg. The inventory of the water distribution system in Winnipeg is summarized in Appendix "B".

In 1990, a total of 2,300 watermain breaks were reported in the City of Winnipeg. In general, almost half of the maintenance crews' time is used in repairing watermains in Winnipeg. The workload distribution of the 11 water maintenance crews by service programs based on actual total man hours used in 1990 is summarized in **Table 2**.

TABLE 2

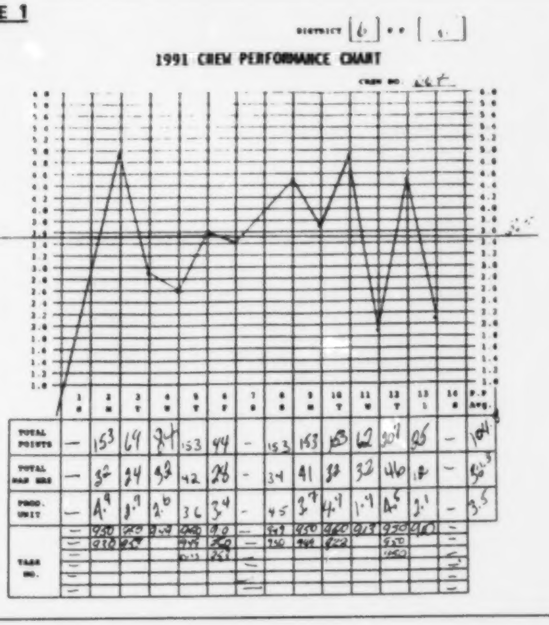
WORKLOAD DISTRIBUTION OF WATER MAINTENANCE CREWS

Service Program	Manpower Distribution
Watermains	46 %
Services	35 %
Valves	11 %
Hydrants	8 %

WATER CREW PERFORMANCE CHART

The productivity measurement method developed is called a Water Crew Performance Chart. It was implemented in all six Districts in November, 1987. Each performance chart covers a timeframe of two weeks, which also coincides with the City's pay period cycle. By using the performance chart, foremen calculate the productivity index of their crews and plot the results on the chart daily. The daily calculation and chart updating process takes less than a minute to complete each day. This simple process enables the foremen to pinpoint how productively their crews perform, and to determine the best use of crews in various maintenance tasks each day. The chart will indicate if their crews' performance has deviated from the normal level, and the results will prompt them to take appropriate action to correct it. A sample of a completed Water Crew Performance Chart is shown in **Figure 1**.

FIGURE 1



OBJECTIVES OF THE WATER CREW PERFORMANCE CHART

The Water Crew Performance Chart is intended as a tool for foremen and management to monitor individual crew performance. It was developed with the following objectives in mind:

(1) To be able to measure and monitor the productivity of each waterworks maintenance crew daily,

(2) To assist crew foremen in doing work scheduling more effectively,

(3) To enhance the awareness of productivity improvement among the management and staff within the Operations Department.

Although the Operations Department presently uses a mainframe-supported Maintenance Management System (M.M.S.) to record City maintenance activities of individual tasks, from a productivity management perspective, the introduction of a productivity measurement system was needed in order to effectively monitor the performance of the waterworks crews. The measurement method was originally developed to address the following weaknesses identified in the M.M.S. (Siu, S., Canadian Public Works Congress & Equipment Show Proceedings, 1990, p. 144).

- The M.M.S. does not measure the productivity of individual water crews. It mainly gives an account of cost and budgetary control for each M.M.S. task on a bi-weekly (Pay Period) basis.
- The results generated from the M.M.S. are not readily available until four days after the end of each Pay Period. This delay drastically weakens the effectiveness of using the M.M.S. as a management tool to monitor work performance.

HOW THE PERFORMANCE CHART WORKS

According to Kendrick (Kendrick, J.W., Understanding Productivity, Johns Hopkins University Press, 1977), the definition of productivity is "the relationship between outputs of goods and services and the inputs of basic resources--labour, capital goods and natural resources." In the performance chart method presented here, productivity is defined as follows:

$$\text{Productivity Index} = \frac{\text{Equivalent Work Points}}{\text{Man Hours}}$$

The productivity of each crew is assessed daily by the crew's foreman, based on how many and which maintenance tasks were completed on that day divided by the man hours required to complete the tasks. No work points are given for incomplete tasks; and the man hours continue to accumulate until the task is completed. For each task completed, the crew is credited with equivalent work points derived from statistical analysis of the historical manpower usage data. The historical manpower usage data is extracted from the Maintenance Management System.

In order to make the performance chart as simple and practical as possible for water foremen to use on a daily basis, the calculation of man hours is limited to water crews only. The support staff of the crews do not enter into the picture, as their hours are very difficult to keep track of since they work on a variety of tasks each day. The 14 water excavation tasks and their respective work points on the performance chart are listed in **Table 3.**

For example, if Crew A has used 8 hours to repair a watermain break on pavement (Task 949 - 84 work points) and a service box (Task 910 - 25 work points) on Monday, the productivity index of Crew A on Monday is calculated as follows:

$$\text{Productivity Index} = \frac{84 + 25}{4 \times 8} = \frac{109}{32} = 3.41$$

If the crews work overtime or after their regular 8-hour schedule, the overtime is treated as straight time in the index calculation so that productivity is not unfairly penalized or misrepresented in the calculation for crews who have to work on weekends or after their regular shift when such circumstances arise.

TABLE 3

Water Task	Description	Work Points
910	Excavated repair to service boxes	25
912	Service excavation & repair over 2"	70
913	Service excavation & repair 2" & under	62
922	Service renewal - 2" & under	82
929	Planned valve installation	80
930	Excavated valve replacement	84
932	Excavated valve box repair	25
933	All excavated valve box repairs	53
942	Excavated hydrant repairs	62
943	Planned hydrant installation	100
944	Hydrant replacement	100
949	Watermain repair on pavement	84
950	Watermain repair on boulevard	69
957	Watermain mini-renewals	2x(ft)

LIMITATIONS OF PERFORMANCE CHART

The performance chart method was designed in such a way so that it could meet the following design criteria:

(1) It must be economical.
(2) It must be simple.
(3) It must be acceptable to the staff who use it.
(4) It must be able to achieve all its objectives.

However, the Water Crew Performance Chart, like many other management tools, has its limitations.

- To keep the calculation simple, the performance chart method is used to measure the productivity of waterworks maintenance crews on excavation tasks only. Non-excavation tasks, such as cut maintenance, are not monitored. It is estimated that over 90 % of waterworks maintenance activities are related to excavation tasks.
- To avoid duplicating cost data presently available in the M.M.S., the performance chart contains no cost data. The chart itself is to be a productivity indicator. The monitoring process and the information gathered in it are to complement the information provided in the M.M.S. for assessing how effectively labour and equipment resources are utilized in waterworks maintenance activities.

RESULTS

Since the Water Crew Performance Chart was implemented in November, 1987 in the Operations Department, the water maintenance crews have made continued productivity improvement, as illustrated in **Figure 2**. The productivity of individual crews is shown in **Figure 3**.

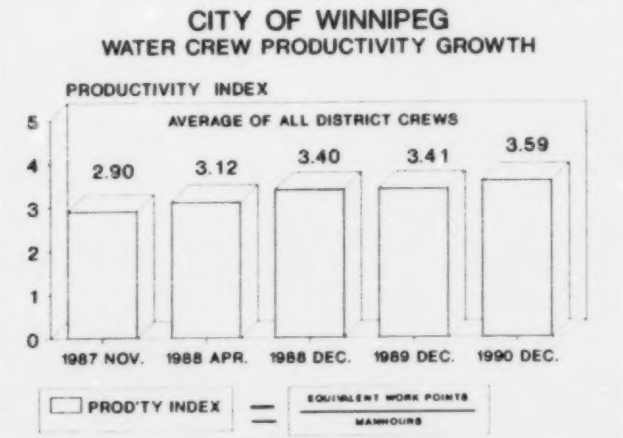


Figure 2 OPERATIONS DEPARTMENT

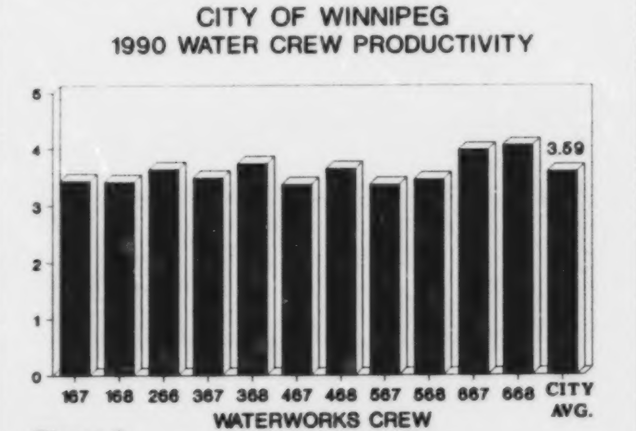


Figure 3 OPERATIONS DEPARTMENT

An overall increase of 0.20 in the productivity index means the crews have increased their productivity by repairing an additional service box (Task 910) in four days using the same resources (labour and equipment). Using 1987 result as a reference base, the cost savings of every 0.20 increase in the productivity index from all 11 crews is estimated to be $150,400 annually (1991 dollars). The breakdown of the cost savings is 70% in labour costs and 30% in equipment costs.

Other positive results that were realized, besides productivity improvement within the Water Sub-section, were:

- The number of completed water excavation tasks was increased from 13 jobs/crew/two weeks in 1987 to 18 in 1990.
- From 1987 to 1990, the man hours required to complete a watermain repair on pavement and on boulevard declined by 16% (**Figure 4**) and 12% (**Figure 5**), respectively.
- Increased awareness of productivity improvement among the staff within the Water Sub-section was a significant intangible benefit.

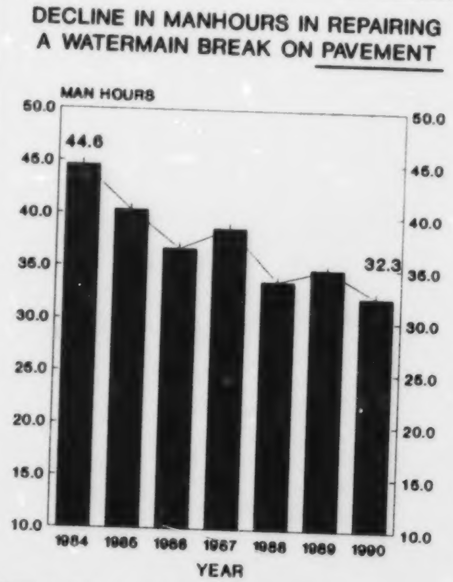


Figure 4 CITY OF WINNIPEG

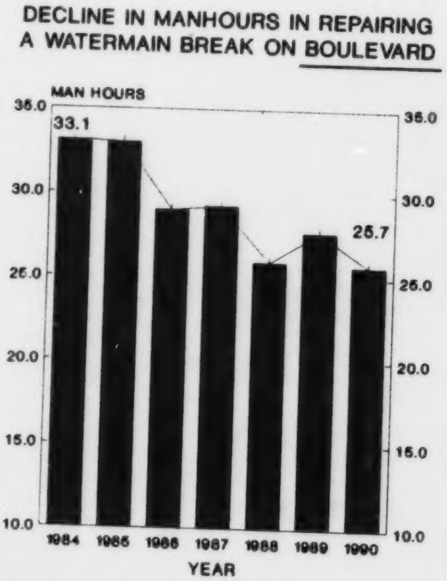


Figure 5 CITY OF WINNIPEG

CONCLUSIONS

The productivity measurement method developed to monitor the productivity of water maintenance crews in the Operations Department has resulted in continued productivity improvement. The many positive results presented above have proved that the efforts contributed by the staff and crews in the six Districts are indeed paying off. The following major conclusions will be of interest to others considering similar approaches:

(1) The use of the performance chart method broadens the database of productivity management analysis. The productivity data that is collected from the Districts complements the information provided in the Maintenance Management System for assessing the effectiveness of using our resources.

(2) The experience learned from the performance chart method is invaluable. It is conceivable that with some modifications and further research work, the method can be adapted and applied to other public works operations as well, such as streets maintenance and refuse collection.

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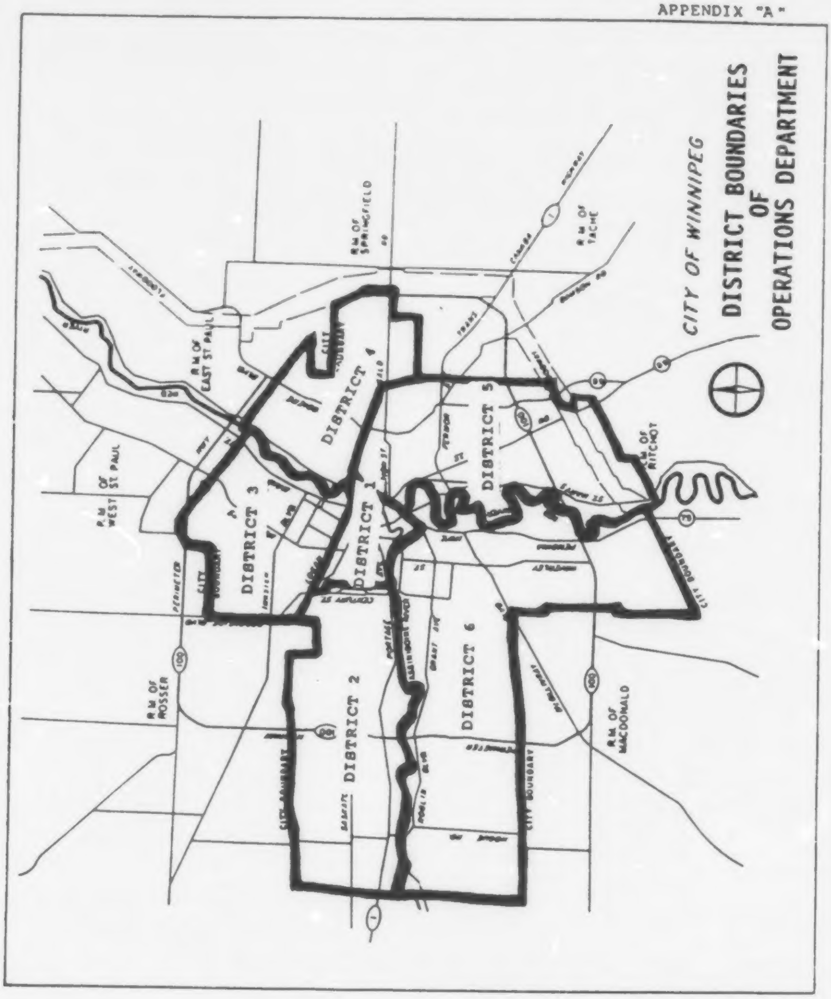
Kendrick, J.W. Understanding Productivity. Johns Hopkins University Press, 1977.

Siu, S. How Industrial Engineering Improves Productivity in Winnipeg. Proceedings of the Canadian Public Works Congress & Equipment Show, Winnipeg, Manitoba, May, 1990, p. 144.

APPENDIX "A"



APPENDIX "B"

1990 City of Winnipeg Inventory of Water Distribution System

Number of Water Meters - 5/8" to 2"	172,751
Number of Water Meters - 3" to 8"	310
Number of Services - 2" and under	175,577
Number of Services - over 2"	3,436
Number of Service Boxes	178,916
Number of Valves - Main	22,490
Number of Valves - Hydrants	17,378
Number of Hydrants	17,391
Length of Watermain (Kilometres)	2,338

WATERMAIN LEAKAGE PREVENTION AND CONTINUING PROBLEMS AFTER REPLACEMENT WITH PLASTIC

Jim I. Yule
President
Colony-Viking Services Ltd.

1. WATERMAIN CORROSION

Most watermain corrosion is evidenced when a main is excavated, exposing the presence of pitting, round concave holes(which are merely an enlarged pit), softness of the pipe (graphitization), and also corroded bolts, rust, etc.

Accelerated watermain corrosion occurred when copper services began to be used 25-30 years ago, but because of the large mass of cast and ductile iron mains connected to a much smaller mass of copper services, it took until approximately fifteen years ago before the first signs of corrosion showed up and which has now accelerated to the point where many towns are contemplating or have begun to replace all of their mains with plastic, removing the "anode", in our opinion, a major error. The corrosion process will shift to an accelerated corrosion of services, and their brass connections, rods, clamps, etc. Many towns that have replaced their mains some years ago are now looking at major problems with their services due to a lack of understanding of what occurs when the cast and ductile iron (anode) is removed.

The Galvanic Series of Metals (Item 1) is your guide to understanding. Any metal more negative in the series if connected to any metal less negative will corrode (cathodically protect) the less negative metal.

The system for prevention of corrosion of ductile and cast iron mains is the same. Ductile iron in the past, has been seen to be an inferior metal due to experiences of utilities where the leakage history on ductile iron is much greater than that of cast iron.

This is due to two main reasons:

1. Ductile iron is normally a thinner walled pipe and therefore a leak would occur at a faster rate than with cast iron.
2. Where ductile iron is connected to cast iron either as a leak repair or as an extension of the system the galvanic series of metals comes in to play and the ductile iron will act as the anode and the cast iron becomes the cathode, in effect the ductile iron will corrode in order to cathodically protect the cast iron.

We have designed a system for effectively preventing further corrosion of cast and ductile iron mains with connected services and now, we had to design a system to protect the services from corrosion that are now connected to the plastic mains that replaced the Cast and Ductile Iron.

2. EXISTING CORROSION PROBLEMS AND CORRECTIONS OTHER THAN MAINS (Figure 1 to 4 and Item 2)

Cast Iron valves, Hydrants, and Tee's isolated in plastic designed systems must be protected from corrosion by sized zinc anodes. Now, coated valves and hydrants are appearing on the market. If these are not also installed including a sized zinc anode then any holidays in the coating will cause the current to escape at that point and therefore concentrating the corrosion process at the holiday and quickly creating a hole in the hydrant, valve, etc.

Item. 1

REPLACING MAINS IS OVERKILL

Galvanic Series Of Metals In Order Of Electrical Energy

(In Moderately Corrosive Soil)

Metals	Millivolts CU/CUS04	Remarks
Magnesium	1700	High in electron energy
Zinc	1200	- - - - -
Aluminum	1000	
Gal. Steel	900	
Steel	700	
Ductile Iron	600	
Cast Iron	500	
Lead	500	
Brass	400	
Stainless Steel	300	
Copper	200	
Cu/CuSO4	0.00 Reference	
Silver	+250	
Graphite	+350	Low in electron energy
Gold	+1130	+ + + + +

COLONY-VIKING

Item 2.

CADWELDING STANDARD PRACTICE

1. Remove all coating material from the area on the main where cadweld is to be placed.

2. File the pipe surface down until a shiny metal appears.

3. Strip about 2" of coating off the end of the wire on the anode. Double this 2" over so you have 1".

4. Make sure mold is clean, dry, and in good condition.

5. Insert steel disk, dished (concave) side up, in crucible to cover tap hole. Dump weld material into crucible being careful not to upset steel disk. Squeeze shot container at bottom to loosen starting material. Place approximately 1/4 to 1/3 of the starting material on the top lip of the mold at cover opening. Place the remaining starting material over the welding material. Close cover.

6. Place wire over shiny metal surface, wire should not be more than 1/2 way across crucible hole, then place crucible over wire. Hold the crucible firmly and aim flint ignitor from side of cover opening. Ignite starting material on mold lip. Do not tap the crucible with the gun because all this does is cause the crucible to move and some of the weld escapes from the bottom.

7. Allow approximately 30 seconds to go by for completion of reaction and metal to solidify.

8. Open and remove mold carefully to prevent chipping of mold.

9. Pull on wire to check cadweld has held.

10. Apply denso paste over and around cadweld.

11. Clean slag (residue) off mold using a natural bristle brush, soft cloth or newspaper before making next weld.

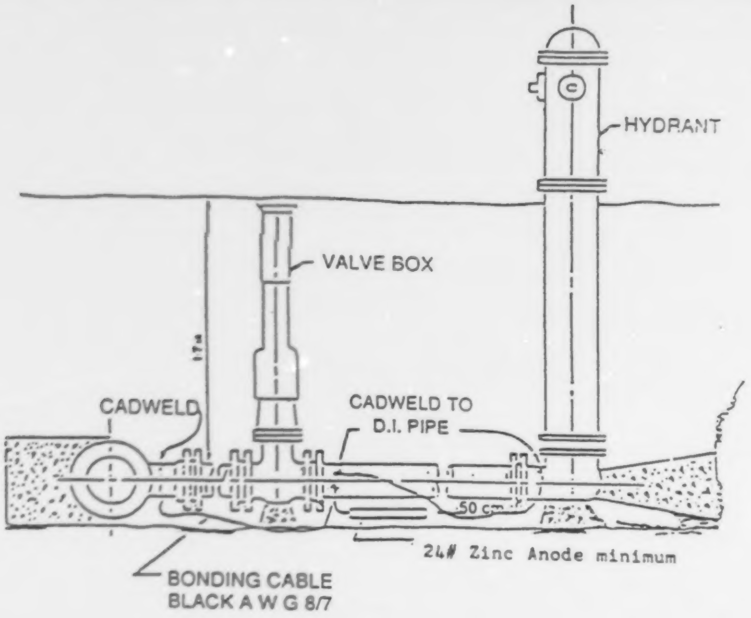
12. Always make sure that the anode wire is tied around the pipe or structure to prevent anode pulling directly on the weld.

NOTE: When storing or transporting cadweld shots make sure they are kept in an upright postion so starting material stays on bottom.

When welding onto cast iron, use a 25g cast iron shot—order number 25PCI.

When welding onto steel, use a 15g steel shot—order number CA15.

Fig. 2.



Install zinc anodes where shown not less than 24#. Wrap all exposed nuts and bolts with mastic or denso paste, especially the bottom 9" of the valve operating rod especially where the yoke connects on to the service stop.

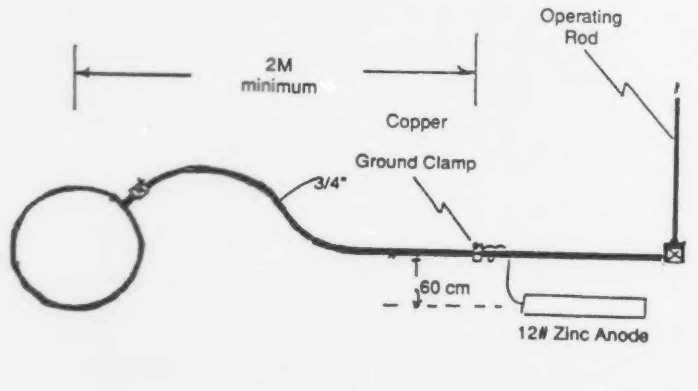
BONDING HYDRANTS/VALVES

Date 01:01:86	Section 5. 6. 03.	Scale N.T.S.
Drwg. No. 9	Drawn by	Chek'd by

COLONY VIKING Services Ltd. P.O. Box 1500. St. Paul, Alberta [illegible]

COLONY-VIKING

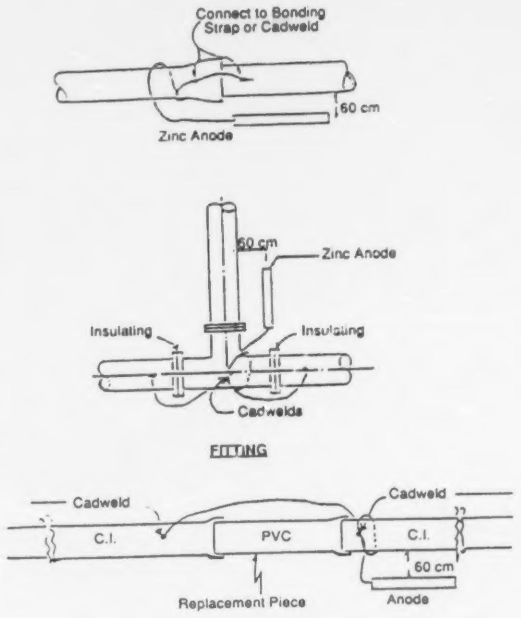
Fig. 1.



SERVICE ANODE INSTALLATION

Fig. 3.

COLONY-VIKING



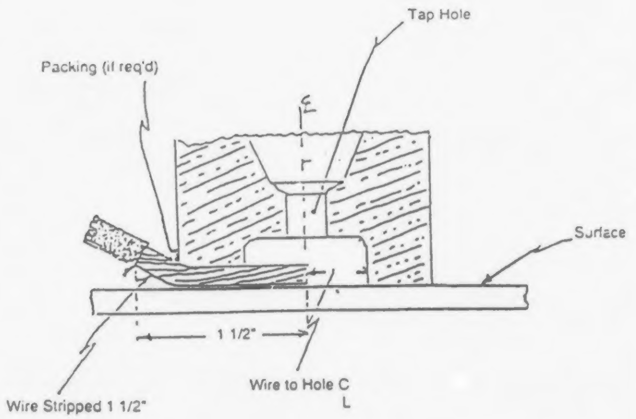
Wrap Anode Wire around and Tie

BONDING MAINS/FITTINGS

Date 01:01:86	Section 5. 6. 02.	Scale N.T.S.
Drwg. No. 8	Drawn by	Chek'd by

COLONY VIKING Services Ltd. P.O. Box 1520. St. Paul. Alberta T0A 3A0

Fig. 4.



When cadwelding, make sure exposed wire is inserted only half way across mould as shown.

Wrap or coat all exposed metal surface with mastic or denso paste.

CADWELDING - CABLE INSERTION

Date 01:01:86	Section 5. 5. 03.	Scale N.T.S.
Drwg. No. 7	Drawn by	Chek'd by

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Preventive maintenance of plastic designed systems must include the following:

1. Steel operating rods should be wrapped with Denso or similar tape or paste to insulate the Y yoke to approximately 12" of the operating rod. This is to ensure that where the rod is possibly in contact with the soil no corrosion can occur.
2. All bolts should also be wrapped or paste applied.
3. Packaged zinc anodes must also be cadwelded to each isolated metal part of the plastic system.
4. A single T.W.H. #10 insulated copper wire should also be run from the below ground metal to the surface for testing purposes.
5. No anode should be installed below ground without some means of determining whether or not the anodes is giving a service life.

Problems preventing a service live can be:

- A. Poorly made anodes
- B. Anodes using poor quality zinc.
- C. Mechanical problems arising from inferior workmanship such as poorly soldered anode rod to the copper wire.
- D. Anode wire not wrapped around the main causing a strain to be placed on the cadweld possibly resulting in failure if the anode sinks in wet soil conditions. See Item 2.
- E. Zinc anodes only should be used on watermain installations. The use of magnesium anodes results in a short end life because magnesium metal will dissolve in water even when it is not attached to any structure.

We cannot suggest the use of stainless steel as a solution either as bolts or as operating rods because the problem of all metal corrosion in the system has not been met when stainless steel bolts or rods are used. There will still be existing clamps, cast bodies, brass stops etc.

We recommend that a zinc anode designed system for existing plastic mains with attached copper services monitored by means of a test post monitoring station is the desired method of installation for protection of services after plastic main replacements.

On the use of zinc caps on bolts and nuts.

Zinc caps provide a temporary protection only. There is a much greater difference in potential between the zinc cap and the bolt, this makes the cap deteriorate at 5 to 7 times the corrosion rate of an ordinary steel cap (1200 m.v. - 750 m.v. in normal soils). Much more protection would be provided if the nuts were coated with mastic or Denso tape. A packaged zinc anode of minimum 24#, should of course be bonded to the body of the valve, flange, etc. This should provide 20-25 years of protection.

3. THE ZINC AND MAGNESIUM CONTROVERSY

Our position in this would be that the utility should include in the invitation to bid, clauses similar to those included in number 3 and 5 of Item 3.

3. The anode life of the system will be for 25 years.
5. Tests will be conducted after installation to establish "asbuilt" readings to ensure design parameter standard of minimum polarization voltage shift of 100 millivolts are met.

Also, the zinc assay of the supplying mine should be obtained (Item 4), along with an independent lab test of the anodes being supplied.

COMMITMENT LETTER

Item 3.

1. Workmanship and materials are guaranteed for a period of five years.
2. Monitoring and testing with reports are provided free for a minimum of four years on the systems we install.
3. The Anode life of the system will be for 25 years.
4. Tests will be conducted prior to installation to establish existing millivolt measurements at all services connected to the proposed main for cathodic protection.
5. Tests will be conducted after installation to establish 'as built' readings to ensure design parameter standard of minimum polarization voltage shift of 100 millivolts are met.
6. Test posts will be installed at each 'bank' of anodes, normally on each block of houses to ensure ease of monitoring.
7. Zinc anodes will be installed to standard specification using only Super High Grade zinc with minimum iron content of .0014.
8. Monitoring will determine the time to replace the anodes to enable the lifetime of the existing watermain to continue.
9. The anodes will be installed in the boulevard area by means of augering to a minimum depth of nine feet with a minimum cover of four feet; each anode being connected to a #8 TWH 90 header cable terminating at a test post for each 'bank' of anodes.

Item 4.

HUDSON BAY MINING AND SMELTING CO. LIMITED
FLIN FLON, MANITOBA
CANADA

Form No. 554

ASSAY AND WEIGHT CERTIFICATE

HUDSON BAY
SPECIAL HIGH GRADE
ZINC

On Shipment Consigned to COLONY VIKING SERVICES LIMITED
4837 46th AVENUE
ST. PAUL, ALBERTA

Date OCT. 22, 1990

H.B.M & S. Order No. 2681-D

Customers Order No.

LOT	SLABS	POUNDS	CAR NO.	MARKINGS	CD %	FE %	CU %	PB %	ZN %
961	360	19,903#	TRUCK	NIL	.0002	.0004	.0012	.0014	99.9968

The above weights are certified to as being a correct copy of Weight as recorded by the sworn Weighmaster.

The above analyses are certified to as being a correct copy of Assay Results as recorded by the Chief Chemist

HUDSON BAY MINING AND SMELTING CO. LIMITED

Per

Zinc - is a metal which will provide adequate protection for any watermain system (Item 5) using our method of installation.

Magnesium - Is a metal which will provide inadequate protection for watermains (Item 6).

To date in all the years that we have been installing cathodic protection systems we have used only zinc anodes. It is possible to obtain current levels up to 1.5 to 2.0 Amps and the polarization necessary, this is sufficient for installations using the block by block system.

Magnesium at 1.7 volts having greater potential than zinc at 1.2 volts in normal soils, the amperage output can be up to twice that of zinc, nevertheless at the depths watermains are laid, there is no necessity for this greater potential (See no. 5 of Item 3).

If the design criteria is for a period of 25 years, magnesium anodes are likely to be replaced three times during this period. So in turn, the cost of magnesium cathodic protection would be three times the initial cost of installation (See no. 3 of Item 3).

It has been our experience where we have seen magnesium used that the life of magnesium anodes can be as short as five years. We took over a system installed in a city in Alberta using magnesium anodes that depleted over a five year period and leaks had began to show up again. We installed a zinc anode system three years ago and their leaks have stopped.

A simple experiment that can be performed is to pour tap water into 2 glasses and take a small piece of magnesium and a small piece of zinc and place one in each glass. Within a few minutes you can observe the magnesium emitting hydrogen gas and corroding while the zinc remains in its original state. The experiment can be extended to include each metal being connected to a piece of copper.

Therefore, the zinc and magnesium controversy over magnesium self depletion aspect and zinc lower potential is resolved by the use of paragraph 3 and 5 of the commitment letter (Item 3).

4. PREPARATION OF CONTRACTS

The following should be included in any tender document. See Item 3 on the next page.

5. INSTALLATION

Criteria

Location of the anodes is extremely important and we have criteria we use in making this decision that include leak history, size of main, soil characteristics, etc. Each service attached to the main is tested for millivolt potential prior to installation and also after installation to insure that not only the main but the services are polarized. Over the line tests are also performed.

The designed method of installation that has evolved is the "block by block" method, using only zinc anodes, connected in series to give max. current output, in various dimensions and weights of anodes (eg. 24#, 30#,36# in three, four, five feet lengths) (Figure 5) with correspondingly varying outputs.

These are installed, an optimal distance apart, either in the boulevard area; under pavement; close to the main (Figure 6). Soils, number of copper services, size of main, leak history, age of main, and other relevant information is obtained from our testing procedures. A decision is made as to the location of the anode bank within the block area.

Kingston

We recently completed a cathodic protection installation in the City of Kingston. It was a very interesting project in that we originally intended to place the zinc anodes on one end of the watermain in the middle of a four lane Highway (Figure 6) close to the main and at the far end of the system. We had proposed originally to install the anode bank in the boulevard areas.

Item 5.

COLONY · VIKING SERVICES LTD.
P.O. Box 1520, St. Paul, Alberta T0A 3A0, Ph: (403) 645-5662, Fax: 645-3780
Branch Office - Marketing, Suite 1840, in Sun Life Place, 10123 - 99 Street, Edmonton, Alta. T5J 3H1, Ph: (403) 423-0873, Fax: 429-4696

Zinc Anodes:

Zinc anodes shall conform to A.S.T.M. B418-80 Type II and shall have the following composition:

Aluminum	-	0.005%	maximum
Cadmium	-	0.003%	maximum
Iron	-	0.0014%	maximum
Lead	-	0.003%	maximum
Copper	-	0.002%	maximum
Zinc	-	Remainder	

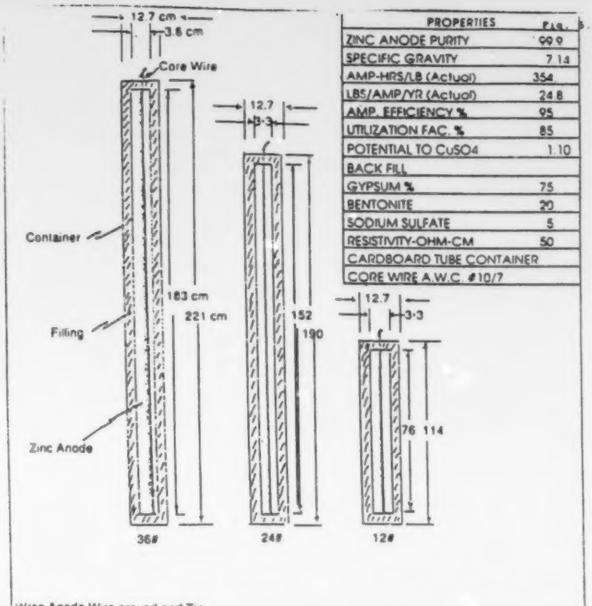
Item 6.

COLONY · VIKING SERVICES LTD.
P.O. Box 1520, St. Paul, Alberta T0A 3A0, Ph: (403) 645-5662, Fax: 645-3780
Branch Office - Marketing, Suite 1840, in Sun Life Place, 10123 - 99 Street, Edmonton, Alta. T5J 3H1, Ph: (403) 423-0873, Fax: 429-4696

Magnesium Anodes:

Magnesium anodes shall conform to the following composition:

Al	-	0.02%	maximum
Mn	-	0.80 to 1.5%	
Fe	-	0.03%	maximum
Ni	-	0.002%	maximum
Cu	-	0.003%	maximum
Zinc	-	0.02%	maximum
Other	-	0.02%	maximum
Magnesium	-	Remainder	



PROPERTIES	Fig. 5.
ZINC ANODE PURITY	99.9
SPECIFIC GRAVITY	7.14
AMP-HRS/LB (Actual)	354
LBS/AMP/YR (Actual)	24.8
AMP. EFFICIENCY %	95
UTILIZATION FAC. %	85
POTENTIAL TO CuSO4	1.10
BACK FILL	
GYPSUM %	75
BENTONITE	20
SODIUM SULFATE	5
RESISTIVITY-OHM-CM	50
CARDBOARD TUBE CONTAINER	
CORE WIRE A.W.C. #10/7	

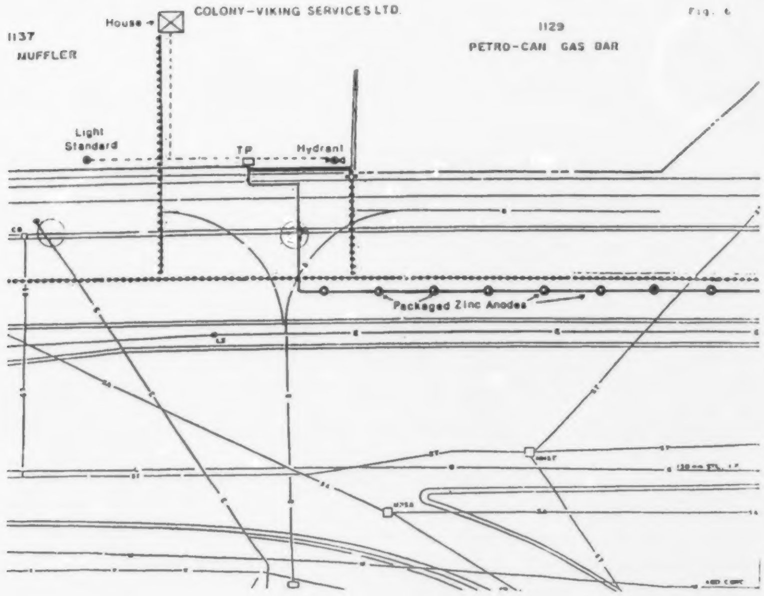
Wrap Anode Wire around and Tie

PACKAGED ZINC ANODES

Date 01.01.86	Section 3. 1. 00	Scale NTS
Drwg. No. 5	Drawn by	Chek'd by

COLONY VIKING Services Ltd. P.O. Box 1520 St Paul Alberta T0A 3A0

Fig. 6



We performed tests of the soil which included drilling to the depths where the anodes would be installed at each proposed anode bank location. However, we were prevented from doing so on two of the locations because of the proximity of a fibre optic cable which required hand installation within 10 ft. of these cables. So two locations were changed to drilling in a centre median area.

Completion of test drilling indicated a wide variety of soils and depths to rock bed. One of the installations having a rock bed five ft. from the surface the other two approximately 10 ft. The soils varied from a light gravel, wet clay, and black loam type soil along the length of the main.

We performed five different types of tests before commencing installation. These included main continuity tests, service connection tests, power ground wire continuity tests, service millivolt tests, and main millivolt tests. All of these being conducted for each bank of anodes to determine location, type and number of anodes required. All of which takes a considerable amount of time.

Where the rock formation was only five ft. deep we did some angular drilling to insure the anodes were installed below any frost level. If anodes are within frost, the following results can occur when installed above the main:

1. Frost penetration will cause the anodes to reduce current output, while corrosion conditions continue to occur at the watermain depths.
2. As soil dries during summer conditions anodes can be similarly affected.
3. Parallel installations are limited in their single anode current output. It was an interesting contract in that to activate the system one of the banks was connected to a customers tap because the ground wire of the electrical system that we had proposed to connect to, did not indicate continuity with the main. Two of the banks were connected to the ground system of the over head light standards and one was connected to a water hydrant so that in this installation it was extremely important that the continuity tests were all proven prior to installation.

6. TEST RESULTS FOX CREEK, ALBERTA/PRINCE ALBERT, SASKATCHEWAN/ KINGSTON, ONTARIO.

Each of these tests (Item 7 and 8) are self explanatory and all were taken within a few days of the installation with the exception of Prince Albert, Saskatchewan (Item 9). The voltage shift can be calculated deducting the service millivolt reading from the as found reading. These are attached as illustrations of the results of three different types of systems.

We monitor these systems for a period of five years and report to the city each year as part of any system installed by us. If discrepancies are found, corrections are made.

7. ENVIRONMENTAL CONSIDERATIONS

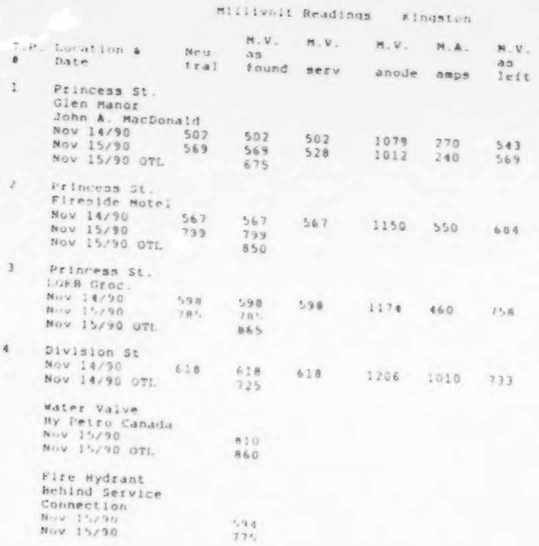
If a high potential anode is used in wet soils, hydrogen is produced and rises to the surface. This can result in:

1. The exclusion of air to the detriment of surface plants and grasses.
2. H2S gas can be produced from some soils resulting in atmospheric production of H2So4 or small traces of acid rain.
3. The "rotten egg odour" is associated with use of magnesium anodes in water heaters. The article in the July 1990 issue of Opflow explains this problem and is good reference material.

8. CONCLUSION

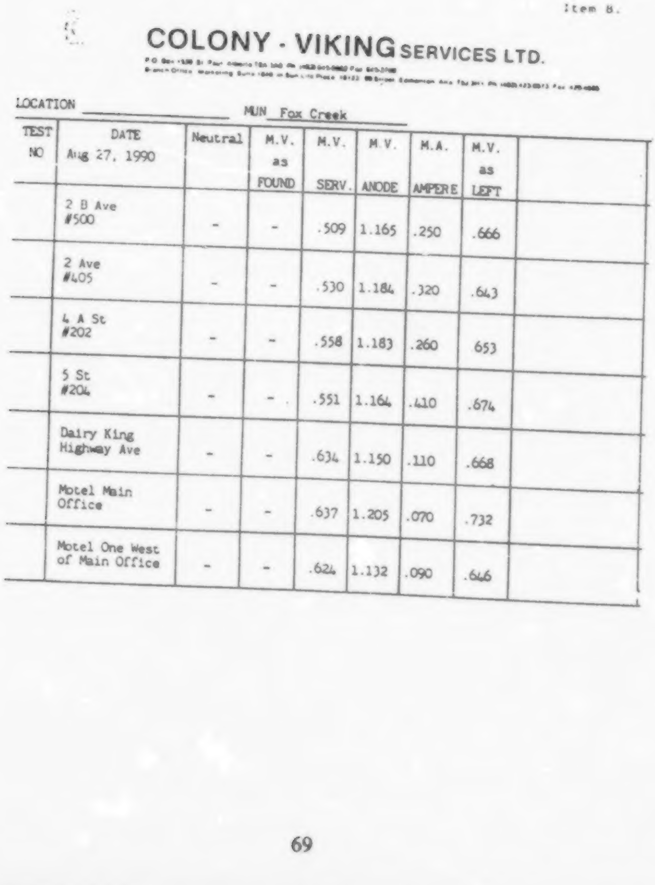
This paper is the result of my own field experiences and a practical view point.

The major mistake made in the prevention of Corrosion of watermains is the application of gas & oil main cathodic protection experiences directly to watermain installations.



Millivolt Readings Kingston

T.P. #	Location & Date	Neutral	M.V. as found	M.V. serv	M.V. anode	M.A. amps	M.V. as left
1	Princess St. Glen Manor John A. MacDonald						
	Nov 14/90	502	502	502	1079	270	543
	Nov 15/90	569	569	528	1012	240	569
	Nov 15/90 OTL		675				
2	Princess St. Fireside Motel						
	Nov 14/90	567	567	567	1150	550	684
	Nov 15/90	799	799				
	Nov 15/90 OTL		850				
3	Princess St. LOEB Groc.						
	Nov 14/90	598	598	598	1174	460	758
	Nov 15/90	785	785				
	Nov 15/90 OTL		865				
4	Division St						
	Nov 14/90	618	618	618	1206	1010	733
	Nov 14/90 OTL		725				
	Water Valve By Petro Canada						
	Nov 15/90		810				
	Nov 15/90 OTL		860				
	Fire Hydrant Behind Service Connection						
	Nov 15/90		594				
	Nov 15/90		775				



Item 8.

COLONY - VIKING SERVICES LTD.

[illegible]
[illegible]

LOCATION ______________ MUN Fox Creek

TEST NO	DATE Aug 27, 1990	Neutral	M.V. as FOUND	M.V. SERV.	M.V. ANODE	M.A. AMPERE	M.V. as LEFT	
	2 B Ave #500	-	-	.509	1.165	.250	.666	
	2 Ave #405	-	-	.530	1.184	.320	.643	
	4 A St #202	-	-	.558	1.183	.260	653	
	5 St #204	-	-	.551	1.164	.410	.674	
	Dairy King Highway Ave	-	-	.634	1.150	.110	.668	
	Motel Main Office	-	-	.637	1.205	.070	.732	
	Motel One West of Main Office	-	-	.624	1.132	.090	.646	

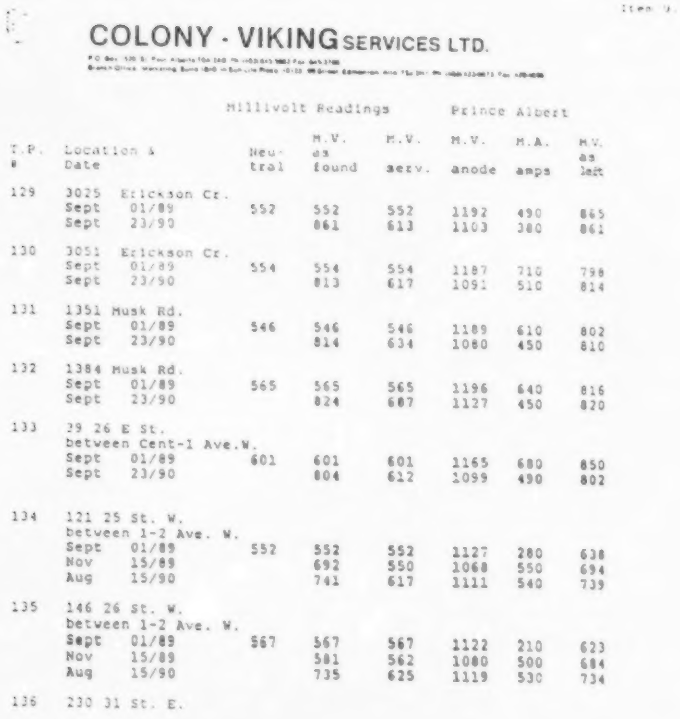
I had to reconsider some gas and oil cathodic protection concepts and have done some pioneering work in this field with the thought that I may contribute to the reduction of the costly replacement of existing cast and ductile iron watermain systems.

Some of the problems were:

1. Gasmains are installed three feet deep; watermains can be up to ten feet deep.
2. Gasmains are normally well coated continuous welded steel mains; watermains have mechanical joints.
3. Gasmains do not have ground wires connected to them; watermains can have power telephone cable etc. grounds to the copper service.

So the problems of Cathodic Protection of watermains is not as simple as the protection of Oil & Gas mains. It is varied and complicated.

We have solved many problems in the past and feel confident that cathodic protection can extend the life of many existing watermains.



Item 9.

COLONY - VIKING SERVICES LTD.

Millivolt Readings Prince Albert

T.P. #	Location & Date	Neutral	M.V. as found	M.V. serv.	M.V. anode	M.A. amps	M.V. as left
129	3025 Erickson Cr.						
	Sept 01/89	552	552	552	1192	490	865
	Sept 23/90		861	613	1103	380	861
130	3051 Erickson Cr.						
	Sept 01/89	554	554	554	1187	710	798
	Sept 23/90		813	617	1091	510	814
131	1351 Musk Rd.						
	Sept 01/89	546	546	546	1189	610	802
	Sept 23/90		814	634	1080	450	810
132	1384 Musk Rd.						
	Sept 01/89	565	565	565	1196	640	816
	Sept 23/90		824	687	1127	450	820
133	29 26 E St. between Cent-1 Ave.W.						
	Sept 01/89	601	601	601	1165	680	850
	Sept 23/90		804	612	1099	490	802
134	121 25 St. W. between 1-2 Ave. W.						
	Sept 01/89	552	552	552	1127	280	638
	Nov 15/89		692	550	1068	550	694
	Aug 15/90		741	617	1111	540	739
135	146 26 St. W. between 1-2 Ave. W.						
	Sept 01/89	567	567	567	1122	210	623
	Nov 15/89		581	562	1080	500	684
	Aug 15/90		735	625	1119	530	734
136	230 31 St. E.						

TRENCHLESS REPLACEMENTS - CITY PERSPECTIVE

T. L. Prince, P. Eng.

City of Calgary
Engineering and Environmental Services Department
Sewer Division
P.O. Box 2100, Station "M"
Calgary, Alberta
Canada T2P 2M5

Throughout Europe and North America, there have been a number of growing concerns for the present utilities to fulfill the demands of the future. The Engineering and Environmental Services Department in the City of Calgary shares many of these concerns for our infrastructure. Demands on the existing systems may be significantly different from the original design conditions. The continual aging of these utilities coupled with the spiralling upgrading and replacement costs, a new emphasis to economize and a political interest to minimize tax increases has placed a greater emphasis on developing effective maintenance programs and finding methods to prolong the utility life much longer than original expectations.

Calgary underwent accelerated rates of expansion from 1951 to 1981. Our operation was more in tune with the development and inspection of the new system with a lesser emphasis on infrastructure maintenance. Over the last ten years we have been taking a serious look at our maintenance programs.

EXISTING SEWER INFRASTRUCTURE

According to City records the first sewer system was started in 1890 in Calgary. The first sewer system in Calgary was a combined sewer system. Storm sewer installation was initiated in 1950 using a 2 year Design storm. The separation of the storm and sanitary sewers continued after 1951. The Sewer Division presently is responsible for the operation of two completely separated sewer systems.

As illustrated by Figure 1, there have been two significant periods of population expansion, 1904 to 1914 and 1950 to 1981 respectively. These high growth periods correspond to periods of major infrastructure expansion for the City. Approximately 10% of the sewers were constructed prior to 1914 and 80% were constructed between 1950 and 1981. Also, during Calgary's tremendous building boom between 1971 and 1981, 40% of the sanitary system and 50% of the storm system were installed. Figures 2 to 6 show trends of the high growth on the utilities after 1950 and the recessionary effect in 1981.

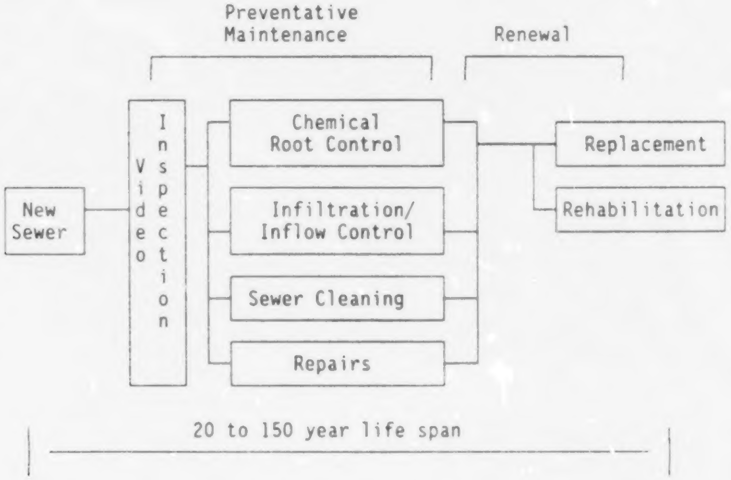
LIFE CYCLE OF A SEWER

The life expectancy (based on accounting depreciation) of the City of Calgary sewers is forty (40) years. Actual experience indicates that most of the materials used for sewer construction last longer than forty years. There are existing sewers installed in the 1800's still in operation. However, some sewers installed in poor soils have been replaced after only twenty years of service.

Studies in England estimated the life expectancies of well-maintained sewers under normal soil conditions as follows:

Diameter Range	Service Life (year)
Greater than 1000 mm	Exceeding 125
300 to 1000 mm	125
Under 300 mm	100

There are two stages, preventative maintenance and renewal, in the life cycle of a sewer.



A sound preventative maintenance program will extend the service life of the existing pipe system and defer the need for new costly capital improvements by conventional replacement. Implementation of new pipe material has appeared as a savior for the problems developed with clay tile and concrete pipe.

The use of polyvinyl chloride (P.V.C.) pipe in diameters less than 375 mm began in 1978 for sanitary sewers and P.V.C. is now the predominant pipe product. However, concrete pipe has been used exclusively for storm sewer installations. The following is an inventory of the major infrastructure components as of 1990 December.

Sanitary & Storm Sewer Infrastructure (to December 1990)

Sanitary Sewer Pipe	2,850 km
Sanitary Service Connections	197,000
Sanitary Sewer Manholes	34,000
Storm Sewer Pipe	2,275 km
Storm Sewer Manholes	29,000
Catch Basins	39,000
Stormwater Outfalls	437

The following provides a summary of significant historical events in establishing the present condition of the sewer infrastructure.

1932	Sanitary sewage treatment plant installed.
1951-1970	Storm and sanitary separation.
1960's	Sulphate resistant concrete pipe was required to high sulphate soils. Major inflow/infiltration in Sunnyside area was remedied by major replacement of Boulevard Trunk and rubber gasket piping specified.
1970's	Sulphate resistant piping required for all underground storm and sanitary sewer pipe.
1978	T.V. video program initiated. Initial uses of PVC for sanitary sewers.
1982	Capacity evaluation on major trunk sewers by flow meters.

1984	Experimentation with T.V. video of service leads. Experimentation with chemical root control. Polyethylene pipe slip-lining was started.
1986	Sewer and Water Inventory Maintenance Management Systems was started (SWIMMS) - digitized database.
1987	Insituform lining technology was started.
1988	Continued slip-liner/insituform evaluation. First module of SWIMMS completed. Contractor video inspection initiated.
1990	Replacement and/or upsizing of existing pipes using trenchless methods.

Most of the new sewers require very little maintenance. As the sewers age and deteriorate, the maintenance requirement increases until it reaches a stage when renewal becomes more cost effective.

The cornerstone to our preventative maintenance program is the use of C.C.T.V. video inspection. Over the last 12 years our main inspection program has responded to demands of the Waterworks and Streets renewal/rehabilitation programs. Recently we have concentrated our effort on inspecting older areas of the infrastructure. To date, our video inspection has covered 35% of the sanitary system (1050 km) and 5500 service connections (2.8%).

The T.V. inspection results are reviewed in conjunction with maintenance and operation histories and a structural assessment of the sewer system to arrive at a plan for preventative maintenance and/or renewal/rehabilitation.

Up to 1973 the annual length of replacement was under 1000 meters. During 1973 to 1983 the average annual length of replacement was approximately 2 km. Infrastructure replacement/rehabilitation has steadily increased since 1984 as denoted in Figure 7. It is expected the actual replacement program will follow the same trend projected by a simple model using two parameters, the average life of a sewer and variance of life span. Sewers with an average life span of 100 years and a variance of life span of 40 years are assumed to be replaced at a uniform rate in a period between 80 to 120 years after initial construction.

Our current budget is shown in Figure 8. If the projections identified previously are a reasonable estimate of future repair/replacement program activities, the budget requirements will double over the next ten years. It should also be noted that the Waterworks main replacement program is currently over $10 million annually.

RENEWAL/REHABILITATION

All operating divisions of the Engineering and the Environmental Services Department have defined Renewal/Rehabilitation programs in the City of Calgary. Structures and Buildings evaluate bridges and buildings, Waterworks have an extensive main replacement program, Streets recondition roads by asphalt overlay and Sewers reline and replace storm and sanitary sewers.

Two important ingredients to an effective Renewal/Rehabilitation program are:

1) The ability to predict the remaining useful life of the infrastructure.

2) Development of an economic program to establish the time to replace the infrastructure rather than to continue to repair. Renewal or replacement by removal and replacing the old with new materials extends the life of the infrastructure only when a total replacement of the link between manholes is undertaken. Rehabilitation extends the life the existing links by adding materials (coating, linings, etc.) to make the infrastructure as "good as new".

Evaluations to renew or rehabilitate are based on the information collected. The utility is evaluated to ensure the system is in service for the maximum practical length of time. It is our philosophy that repairs dealing with localized problems keep the local system in operation; however, having little impact on the service life of the entire system. Analogous to "a chain is only as strong as its weakest link".

The time and resources required to establish the remaining service life of the utility will vary significantly with the accessibility of the infrastructure. It seems logical that surveying the utilities on the surface having high accessibility is considerably easier than excavating through three metres of soil to look at a deep utility. Remote TV investigation

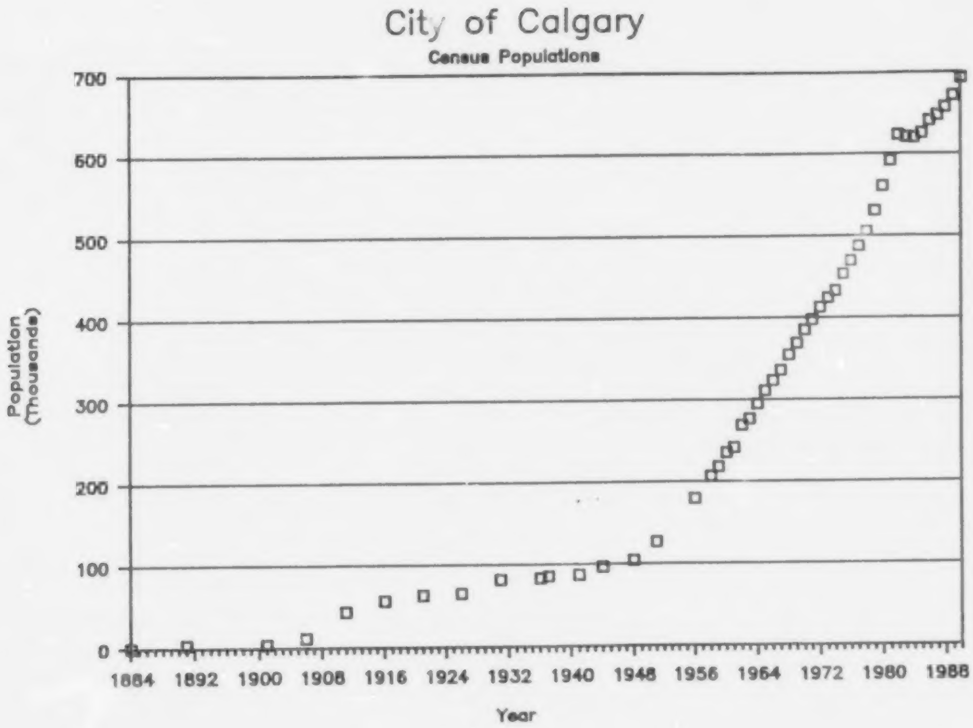


Figure 1

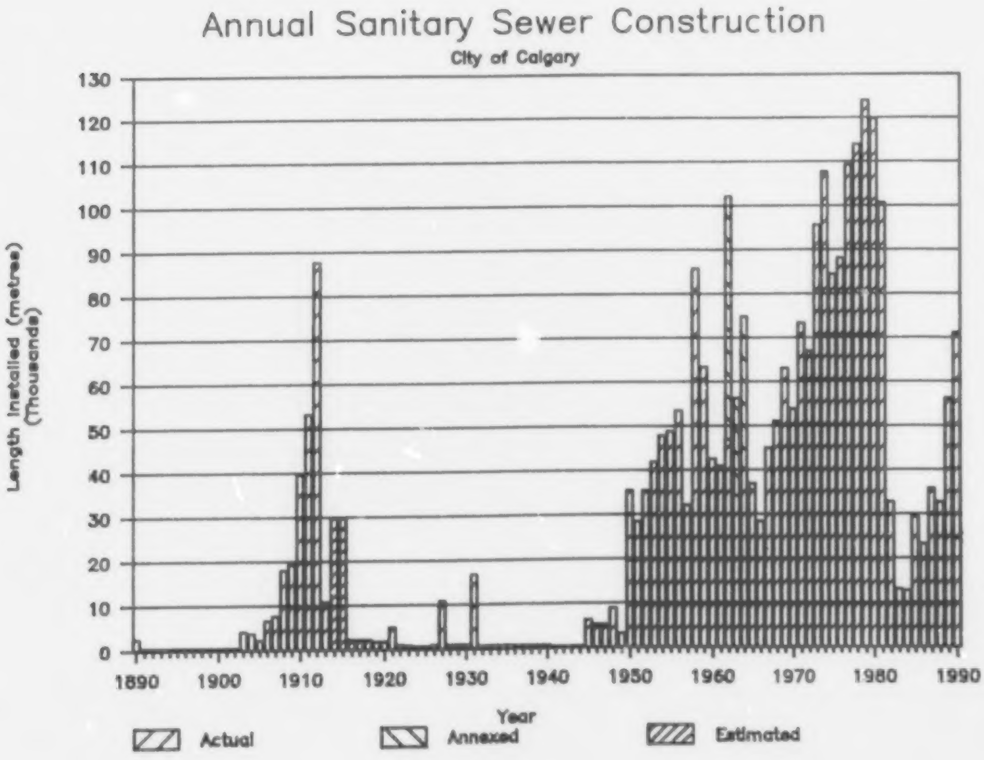


Figure 2

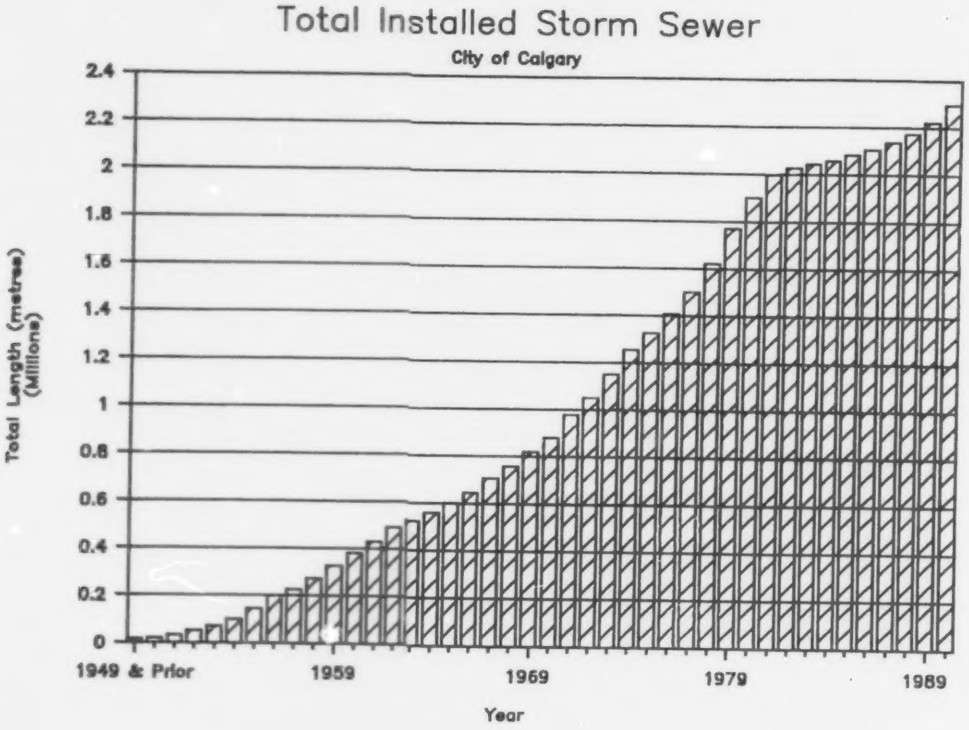


Figure 3

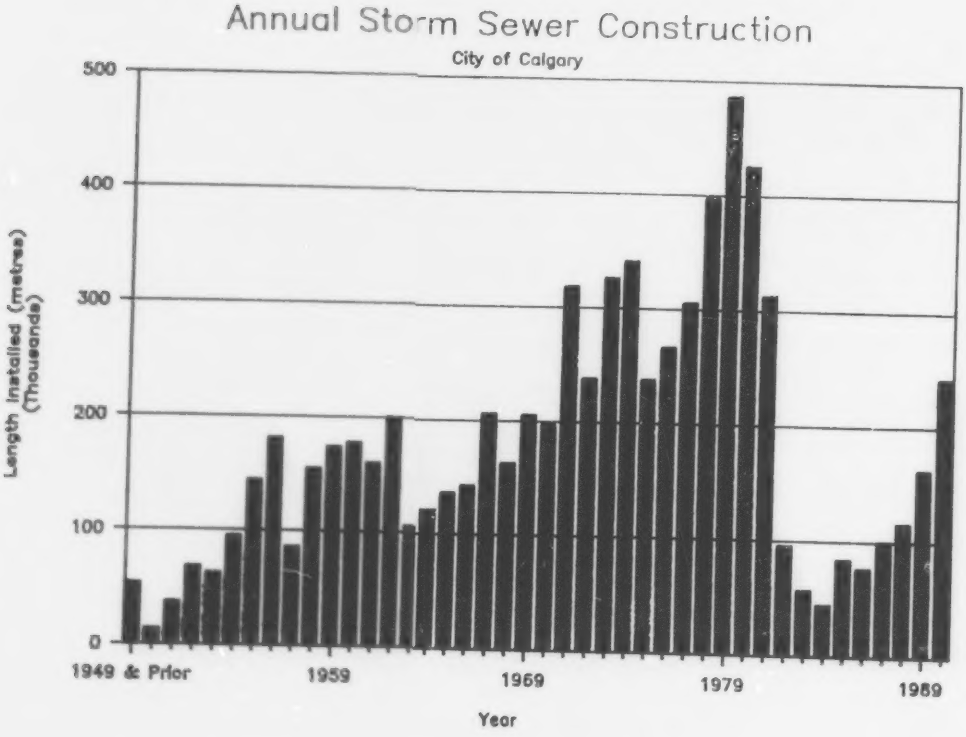


Figure 4

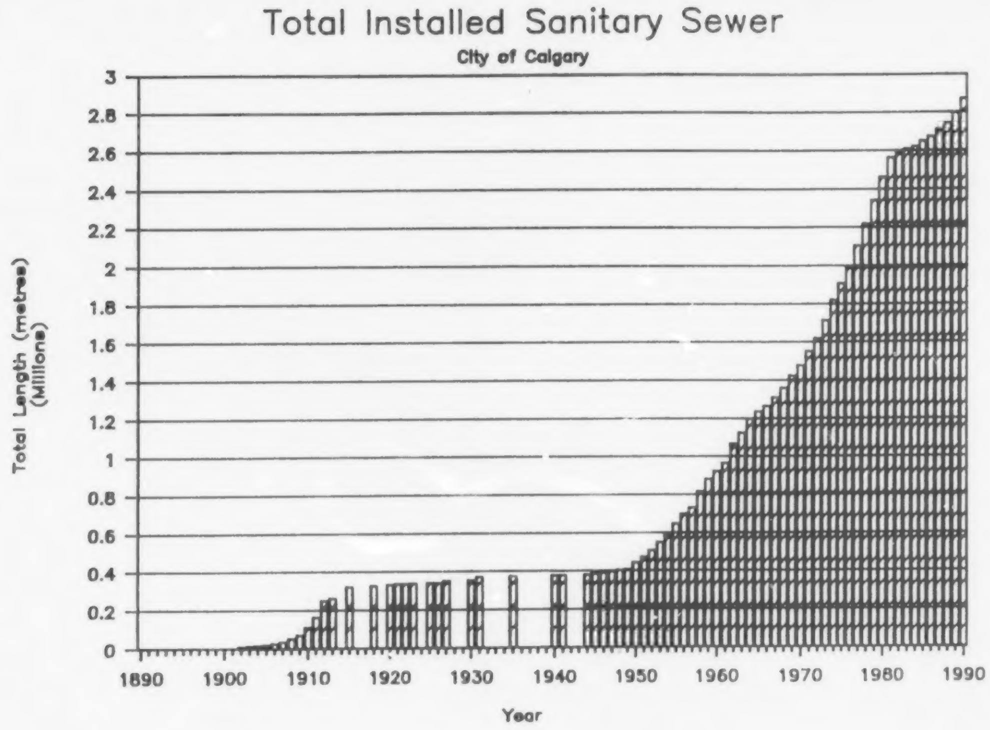


Figure 5

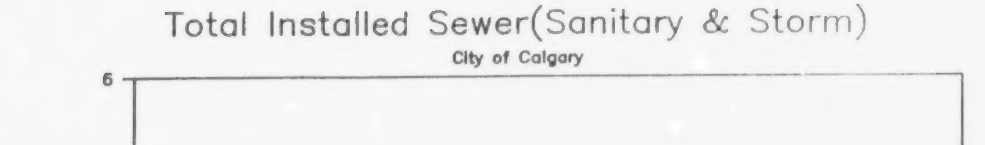


Figure 6

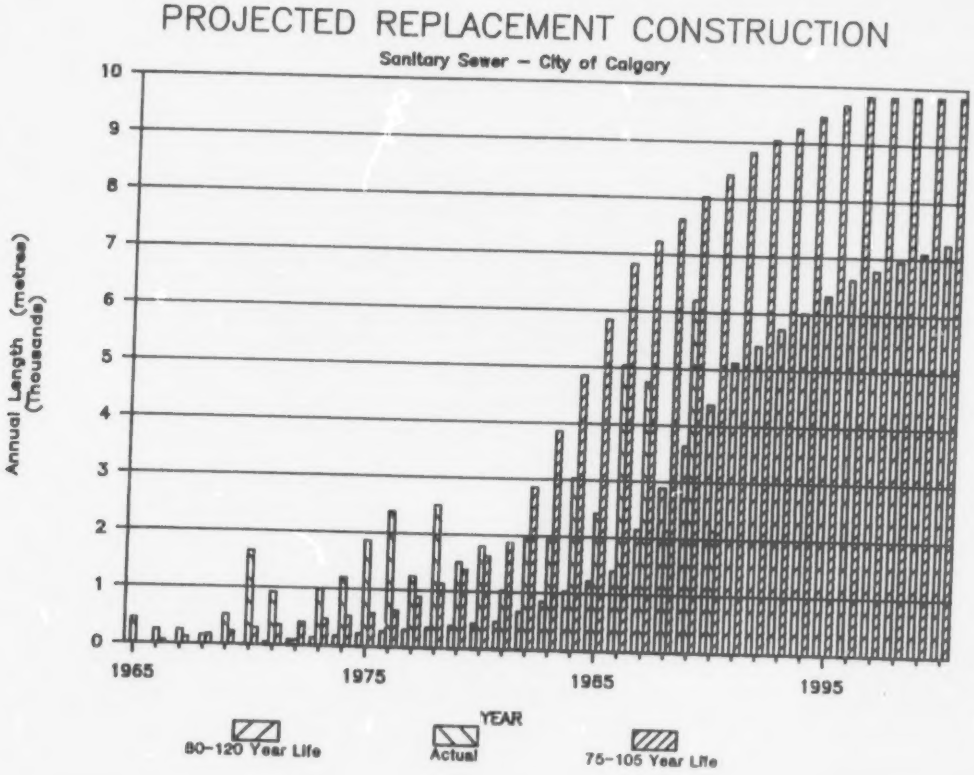


Figure 7

CURRENT BUDGET

Repair of Sanitary Mains	$ 1,200,000
Main Replacement Program	1,800,000
Repair / Replacement of Sanitary Services	2,000,000
Repair of Storm Mains	400,000
TOTAL:	$ 5,400,000

Figure 8

SANITARY SEWERS

INSITUFORM LINERS INSTALLED

YEAR	LENGTH (m)	SIZE (mm)
1988	206	200
	203	250
1989	412	200
	165	250
	202	300
	186	375
1990	122	375

Figure 9

SANITARY SEWERS

SLIP LINERS INSTALLED

YEAR	LENGTH (m)	SIZE (mm)
1984	44	200
1985	98	200
1986	854	200
1987	153	150
	167	250
	304	450
1988	34	150
	34	355
	488	200
1989	28	200
	760	710
1990	290	200
	172	400
	201 *	250

* pipe-bursting technique used

Figure 10

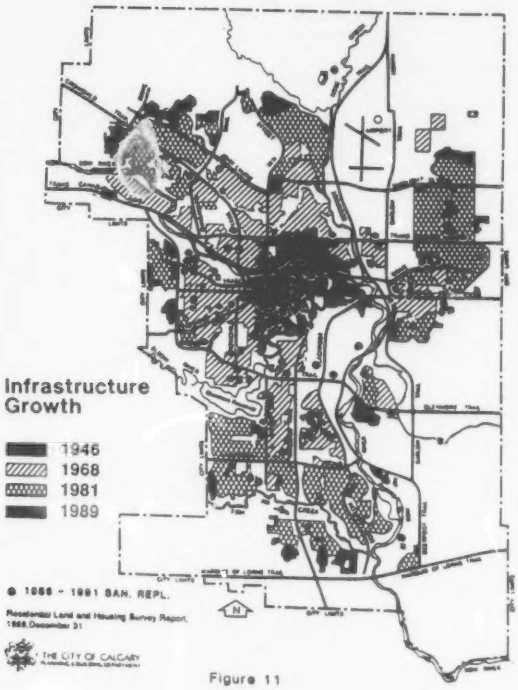


Figure 11

1-L-37D

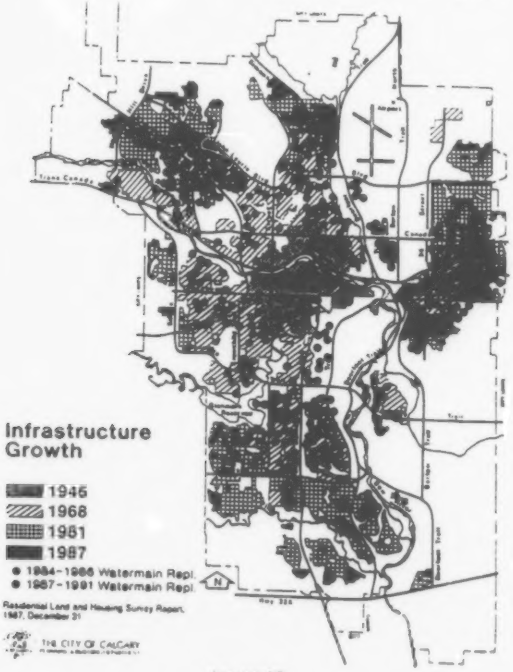
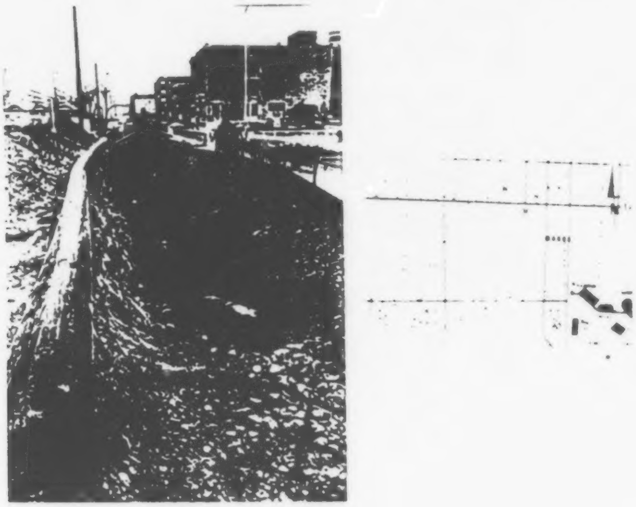

Infrastructure Growth
1946
1968
1981
1987
● 1984-1986 Watermain Repl.
● 1987-1991 Watermain Repl.
Residential Land and Housing Survey Report, 1987, December 31
THE CITY OF CALGARY
N
Trans Canada
Hwy 22X

Figure 12

1-L-37C

10th AVENUE & 2nd STREET S.E. SANITARY SEWER SLIPLINER



CONTRACTOR Bachand Enterprises Ltd

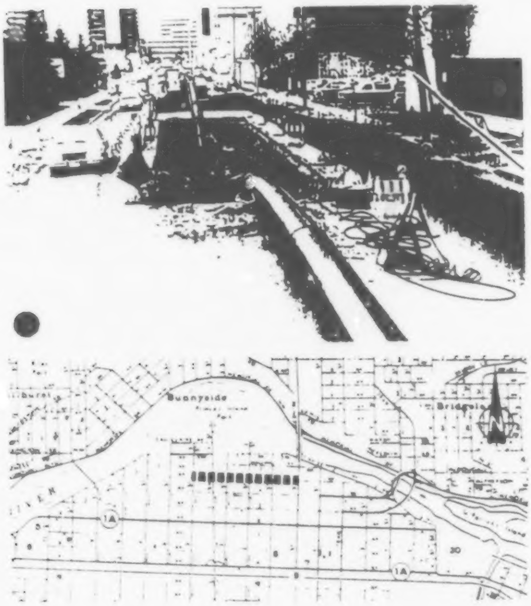
WORK COMPLETED 304m 450mm polyethylene slipliner

PURPOSE To replace the old 500mm clay tile pipe which had deteriorated to the extent that services to customers would have been disrupted

Figure 13

3 rd. AVENUE S.W. DOWNTOWN

SANITARY REPLACEMENT



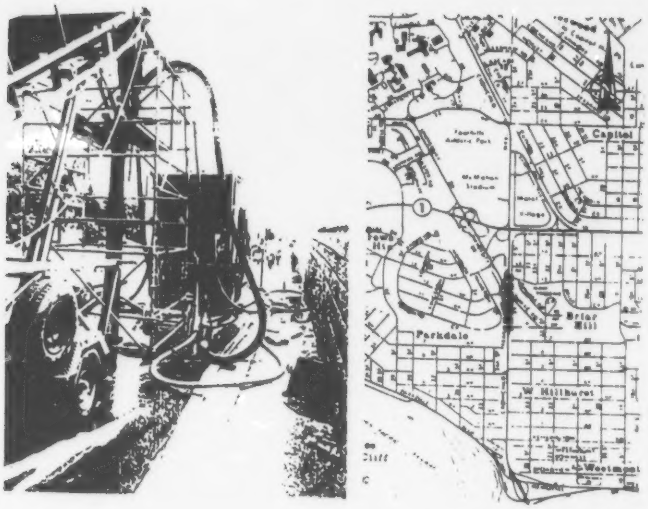
CONTRACTOR Barjac Construction Management Ltd

WORK COMPLETED APRIL - JUNE 1989
760m of 710mm P.E. liner pipe
2 - Sanitary sewer diversion manholes

PURPOSE Rehab existing sanitary sewer, by sliplining process, due to deterioration.

Figure 14

CROWCHILD TR. N.W. SANITARY SEWER REHAB



CONTRACTOR IGL Canada Ltd

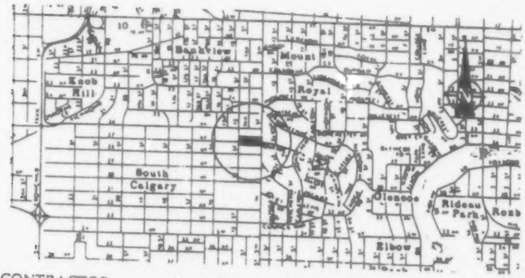
WORK COMPLETED JUNE 1989
196.5m of 300mm insituform liner
185.8m of 375mm insituform liner
16 service reconnection

PURPOSE To rehab existing sanitary sewer system with insituform lining due to deterioration of main

Figure 15

SANITARY SEWER REPLACEMENT
IN LANE SOUTH OF 27 AVE. S.W. & WEST OF 14 ST. S.W.

. 47. E W BY INSERTION LIT. Nov. /90.



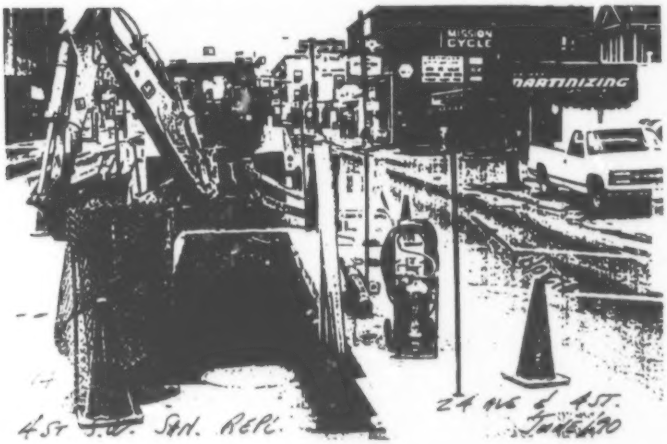
CONTRACTOR : Kerna Construction Ltd.

WORK COMPLETED : November 30, 1990
172 m of 400 mm PE sanitary sewer
4 service reconnections

PURPOSE : To replace an existing deteriorated 525 mm sanitary sewer.

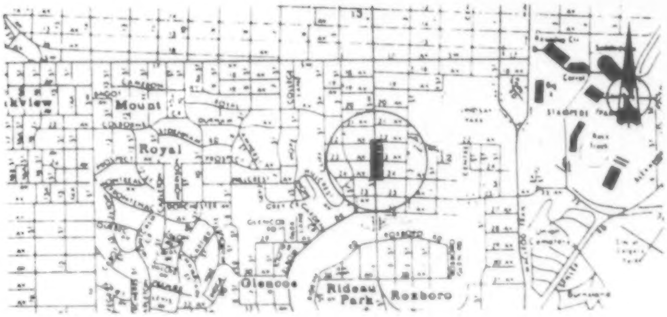
Figure 16

4 ST. S.W. SANITARY SEWER REPLACEMENT



4 ST S.W. SAN. REPL.

24 AVE & 4 ST.
JUNE/90



CONTRACTOR : Caliber Systems Ltd.

WORK COMPLETED : July 1990
200.5 m of 250 mm PE SDR 35 sanitary sewer
6 service reconnections

PURPOSE : To replace an existing deteriorated sanitary sewer between 22 and 24 Ave. S.W.

Figure 17

give us some information but doesn't complete the entire picture as far as the inside structure strength is concerned. Not only is it more difficult, it is considerably more expensive to establish your underground surveys. We view the inside of the pipe; however, we cannot see the exterior without excavation. Alkali attack at the soil/pipe interface may be extreme while the interior may seem mild.

It is necessary and important for municipalities to be able to plan for the most effective renewal, not only to prioritize the most needy, but to estimate what year the item reaches its anticipated design life and what year it should be replaced. If the municipality has an established and reliable data base and can successfully predict what year the utility must be replaced, then an excellent long term renewal plan and budgeting program can be met.

The majority of our renewal and rehabilitation is focused on our sanitary system. The first sewer systems were constructed prior to 1900, with significant level of sewer construction occurring in 1950 to 1981. During modern times sewer expansion occurs during major growths in population. Trends for population growth and sewer expansion were shown in Figure 1 and Figure 6 respectively.

Figure 11 illustrates the relationship between the infrastructure aging and the location of sanitary replacements.

The majority of the replacements are in the areas where the utility was originally constructed more than twenty (20) years ago. With the greatest majority in the area of the city which exceeds forty (40) years of service life. The replacements in the northeast corner of the city are noted exceptions. Our Waterworks Division have soil resistivity readings that correlate with the actively corrosive soils. Figure 12 illustrates the trends of corrosive soil attack on watermains in the northeast quadrant of the city. We are experiencing alkali attacks on our sewer mains in the same areas and therefore do not follow the trends of the aging criteria being established for replacement in the central area of the city.

Our approach to identifying renewal/rehabilitation is based on T.V. inspection. We have six sewer districts in the City of Calgary. The district foreman compile a list of potential replacements based on complaints received, service lead problems, cleaning frequency, main breaks and plugs. Their recommendations are reviewed and T.V. inspected by experienced senior staff. Renewal/Rehabilitation work is then prioritized with other work coincident with Waterworks main replacement programs and Streets resurfacing programs. Figure 8 shows our current budget and we utilized $1.8 million year in our main replacement program.

Up to 1973 the annual length of replacement was approximately 1000 meters. During 1973 to 1983 the average annual length of replacement was approximately 2 km. Infrastructure replacement/rehabilitation and development has steadily increased 3.0 kms in 1984 and 4.4 kms in 1985, 4.5 kms in 1986, 4.2 kms in 1987, 5.6 in 1988, 3.0 km in 1989 and 5.6 kms in 1990.

The renewal/rehabilitation program involves replacement by physically renewing the deteriorated utility with a new pipe of the same size or a larger diameter; by opening a hole and replacing the pipe. Rehabilitation has been completed by three techniques in the City of Calgary, slip lining, pipe bursting and insituform lining. A very small percentage of sewers needing renewal are rehabilitated as particular conditions in the main may make it impractical to rehab. Service leads needing renewal, too many collapses in the main to allow lining, too numerous service reconnections for slip liner, pipe diameter, increased flow; are all criteria to be evaluated in decision making.

Figures 9 and 10 summarize our total installation to date using no-dig alternatives. We are encouraged by other techniques becoming available each day and have provided a number of examples of the different trenchless technologies we are employing for rehabilitation as well as renewal (Figures 13-17).

We have successfully installed eleven insituform liners. Slip liners utilizing polyethylene pipe has had a wider application as a rehabilitation technique. Cost savings of 25% to 75% have been realized by applying rehabilitation methods over total replacement methods.

CONCLUSION

The key activity to our maintenance and renewal/rehabilitation programs is the utilization of T.V. video records. T.V. inspections identify the degree of deterioration of the utility for our decision making process to renew or rehabilitate. Most renewal is restricted to older clay pipe and to where corrosive soils are acting on concrete pipe. The majority of replacement/rehabilitation has occurred in areas of the city that exceed 20 years in age. Approximately 80% of the present sewerage was installed in a 30 year residential expansion period between 1951 to 1981. Consequently, the majority of the system is about the same age and is reaching a median age of 20 years. Structural deterioration is taking place, the potential for a large percentage of the system to require renewal/rehabilitation at a time in the future is significant. The Sewer Division is in the preliminary phase of developing modernized maintenance and renewal/rehabilitation for the Sewer infrastructure. The following conclusions will help in that development.

1) Older areas of Calgary are key areas for developing renewal/rehabilitation techniques in preparation for the future programs.

2) The Sewer Division pipe bursting used recently developed technologies such as slip-lining and insituform lining successfully for sewer rehabilitation and is continuing to monitor its serviceability.

3) We have a full time video inspection crew that has been key to successful implementation of new technologies for sewer renewal/rehabilitation. The development and improvement of this area will benefit our data base development for future time to future analysis.

4) The video inspection program verifies the areas in need of preventative and corrective maintenance as older well known trouble areas of the city.

5) Preliminary long term analysis indicates a need to continue renewal/rehabilitation program to control existing demand and meet future needs.

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AUTOMATION IN THE WASTEWATER INDUSTRYWITH COMPUTER CONTROL

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Fischer and Porter (Canada) Ltd.

HOW HAS COMPUTER CONTROL EVOLVED

In order to answer this, we are going to briefly review more familiar types of control. The first type of control was an indicator. These were mounted adjacent to the production facility. Examples of these are thermometers, manometers and pressure gauges. On the basis of their readings hand valves, levers and dampers were adjusted to change process conditions.

As it became obvious that there were advantages to having a number of indicators in one place, transmitters were developed. A panel was used to provide a surface on which indicators could be mounted. This concentrated the indication of process conditions. Next, recorders were invented so that the history of operating conditions during previous operating periods were available to management as well as to the operators. Recorders took the form of both round chart and strip chart types.

The next step was the development of transmitters, so that a pneumatic signal could be developed and sent to a controller in the process area. Varying as a function of deviation of process variable from the set point, a signal could be developed by the controller to move a valve or other regulating device. Often the controller was combined with the recorder in a huge case. This also was mounted on the control panel.

After the war, through the early 1950's, electronic controllers were developed. These became 6 inches high by 6 inches wide or even smaller. The major portion of the electronics of this controller, however, was mounted in a separate housing in another rack that may be across the room or even in another part of the plant.

The next step was to use digital computers for control. Computers were used in three ways. The first was data acquistion and record keeping. The second was called supervisory control. Signals were sent from digital computers to analog controllers to change set points to values computed by the computer. The third form was direct digital control, more commonly called DDC.

In the 1970's there was another giant technological step. This was the development of LSI or large scale integrated circuits. The shrinking size of electronic components to the point where many circuit devices can be combined on a small silicon chip. This brought prices down, increased reliability and changed the entire world in ways that we are only beginning to appreciate.

COMPUTER SCREEN DISPLAYS

Three displays are used almost universally. These are the overview display, the group display and the detail display. Other displays that are used by almost every supplier but appear with some variation in arrangement and content are trending displays, graphic displays, alarm displays and input value displays.

The overview display accomplishes the same function as the graph of deviation from setpoint but a number of group points are displayed at one time. Eight or more groups may be displayed in an overview and there may be several pages of these. Several points will be positioned along a line, each representing an analog function or a digital function in the group. The analog points will usually be PID controllers. Whether or not the process variable is deviating from the set point of the controller will be indicated by a projection up or down from the line, at the position along the line corresponding to the controller. If the deviation is positive, that is, the process variable is greater than set point, the projection will be upward. If the deviation is negative, the projection will be downward.

The group display will show information about the points in a group of functions in the same detail as the front plate of an analog instrument. In fact, the group is assembled on the CRT and can be made up of the specific complement of funtions that go together to control a portion of a process. Depending on a particular supplier's design, a group may have from twelve to sixteen functions in it. They may be of analog or digital function. In the group display they will be graphically shown, including in the picture the details that are of immediate interest to the operator. The function of a PID controller, for example, frequently shows the process variable and output values as bar graphs, with the set point as an arrow head or pointer positioned along the PV bar at the positon corresponding to the engineering unit values of process variable, output and set point will be dislayed below or to the side of the bar graphs.

From the overview display, the operator can often select a particular group display for viewing by a single key stroke. In some systems, if an alarm condition appears on an overview display, when the alarm is acknowledged from the keyboard the display of the group in which the alarm has occurred will appear on the screen, taking the place of the overview display. In the same manner, acknowledging an alarm in a group display will often call up the detail display of the function in which the alarm condition has occurred.

The third commonly used display is the detail display. It is specific to a single function and shows all the parameters that are pertinent to the function. Often the same bar graph "target" will be part of the detail display, but there will be additional information defining input data, constant and variable parameters, limits and ranges.

There are two types of trend displays. These are real-time and historical. Real time trending shows a record of variable value change for short periods of perhaps two minutes to as long as thirty hours. Historical trending records are for periods of a day or as much as a week. For the trend displays that have a time base of a length such that the period of time between individual points is measured in minutes or hours, the information collected during the period will be averaged, and the average value displayed.

Another type of display, called a graphic display, uses a combination of lines and shapes and alphanumeric characters to draw a picture of a portion of the process. The alphanumeric characters are used to display words as well as values of process parameters such as set points, process variable status, and output values. The graphic picture display may be interactive, that is the process values are real-time values, and are constantly updated. Portions of the display can be made to change shape or color as discrete events take place in the process. For example, when a conveyor is running, its picture may be one color and a label next to it will read ON. When the conveyor stops, the color will change, and the label will read OFF.

Some displays are not graphic but contain information in tabular form. The input display for analog values, for example, will usually be a listing of the types of display associated with the centralized computer control room installation. This tabulation will identify the input by tag name, show the current value in engineering units, and show the range of the variable. In addition, the capability of specifying high and low limits for a process variable, which can be used to initiate alarms may also be available.

Alarms are often displayed in ways that are a combination of graphic and alphanumeric. An alarm condition, with or without an audible annunciation to attract the operator's attention, may be indicated on the screen by a backlighted alphanumeric code or printing of the tag name of the offending point. It may be indicated on an overview by the projection representing the deviation changing to red, and perhaps blinking. It may be indicated by the entire background of the target area of a group display graphic changing color. In addition, a record of alarms may be permanently retained by being typed on a printer external to the display device. It can be temporarily stored as a line on an alarm summary display or on a tabular display recording the most recent twenty or thirty alarms.

OPERATION OF THE BANFF TOWNSITE SEWAGE TREATMENT FACILITY

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Reid Crowther & Partners Ltd.

1.0 INTRODUCTION

In 1985 Reid Crowther and Partners Ltd. were commissioned to determine the most economical means of upgrading the Banff Townsite Sewage Treatment Facility (BTSTF). During the selection of the treatment process, close attention was paid to the aesthetic and environmental impact of the new facilities. In particular, it was essential to minimize the visual, audible and odour intrusion of the plant into the surrounding National Park.

Following a review of many alternatives, it was decided that the plant should provide conventional activated sludge secondary treatment and UV light disinfection. Furthermore, sludge from the plant would be thickened, stabilized by autoheated thermophilic aerobic digestion and then dewatered. Following composting, the dried sludge would be suitable as a soil amendment within the park.

This paper outlines the operation of three processes within the BTSTF; Aeration and Dissolved Oxygen (DO) Control, UV Disinfection and Composting. Following a brief description of each process, the operation of each process is explained. While this paper does not serve as a operating guide to the aforementioned processes, it does serve to inform the reader of the operator's experiences at the BTSTF since 1989.

2.0 DISSOLVED OXYGEN CONTROL

INTRODUCTION

An integral factor leading to the successful operation of an activated sludge process is aeration and DO control. Proper aeration and DO control allows a natural conversion of soluble and non-soluble organic material in primary effluent to CO_2 or additional cell mass ready for secondary clarification. Initially, aeration mixes and adds dissolved oxygen to the effluent mixture; suspending the sludge solids and sustaining microorganism life respectfully. Microorganisms feed on organic materials suspended throughout the tank and in turn produce CO_2 and additional cell mass. Moreover, this desirable microorganism activity increases the quality and settleability of activated sludge.

Controlling every aspect of the aeration process is essential to the effectiveness of any activated sludge system. Over aeration may result in wasted energy and the breakup of biological floc. Conversely, too little aeration will reduce the DO concentration. This restricts the growth of favorable microorganisms in favor of filamentous organisms which in turn adversely affect the settleability and quality of the activated sludge. Therefore, proper control of the aeration system must be achieved for maximum effectiveness.

AERATION SYSTEM

At the BTSTF 3 blowers supply air through the main process air header to a distribution header at each of the four aeration basins. All of the air blowers are similar and are sufficiently equipped to deliver air in response to air pressure levels in the main header. A pressure sensor in the main air header measures the pressure, compares it to a setpoint and then transmits a signal to modulate the valve position in one, two or three of the blower lines, corresponding the supply of air to a set pressure point. Several safeguards, including operator alarms and automatic fail-safe controls, ensure against surge conditions. Typically, two blowers operate during peak periods while one blower operates during low demand.

Four separate headers supply air to Aeration Basins 1A, 1B, 2A and 2B. The lines contain a flow control valve to proportion airflow between each manifold. The manifolds convey air to the course bubble non-clog diffusers placed evenly across the entire floor of the aeration basin. Each main air flow control valve automatically controls the amount of air entering a particular aeration basin, corresponding to the basin's DO concentration setpoint. An oxygen probe measures the DO, compares it to a setpoint DO and modulates the air flow control valve in the basin line accordingly.

OPERATION

The BTSTF Aeration and DO control system requires minimal maintenance by the operator. The Aeration/DO control system simplifies into 2 steps for the operator under normal conditions. First, the operator quarterly compares DO meter readings to the manufacturer's guidelines. Second, the operator annually calibrates and changes the DO sensors. In general, the DO control system is sufficiently automated to provide adequate service under most conditions with little modification left to the operator.

However, two problems occur which require minor operating modifications to improve the performance of the Aeration/DO control system. First, a minimum airflow situation is likely to occur in the second basin of each train. Second, peak/shoulder periods coupled with filamentous bacteria cause problems of sludge bulking and poor settleability.

During periods of low organic loading in the influent, the first basin (1A and 2A) of each train handles a majority of the organic loading. Remaining is a relatively small organic load for the second basin. This results in a situation where the automatic controls may completely shut off air flow. Fortunately, the valves are restrained by a mechanical limit stop, thus preventing a stoppage of turbidity in the second basin of each train. Furthermore, to correct the situation, a minor process alarm alerts the operator to either bypass some of the influent flow to the second basin Process (1B or 2B) in each train and or lower the DO setpoint of the first basin in each train. Likewise, the Process Management System (PMS) may automatically modulate the flow control valves.

Given peak/shoulder periods and filamentous bacteria in the influent, the operators of the BTSTF reverse the DO setpoint concentration on the parallel basin trains. During the summer months the operators change the DO concentration setpoint in the first basins of each train to a lower number (1A and 2A = 2.0 mg/L) than the second basins of each train (1B and 2B = 3.0 mg/L). Although unorthodox, this modification reduces the likelihood of minimal flow situations in the second basins of each train and ameliorates a situation of poor settling and sludge bulking caused by filamentous bacteria in the summer months.

3.0 ULTRA VIOLET LIGHT DISINFECTION

INTRODUCTION

There is a growing trend towards Ultraviolet (UV) light disinfection as a means of inactivating pathogenic bacteria in wastewater. A UV light disinfection system's effective and economical operation makes it a viable alternative to ozone and chlorine use.

The UV disinfection process is quite simple and relies upon the germicidal effects of UV light known since the late nineteenth century. Essentially, UV disinfection consists of treated wastewater flowing through banks of UV lamps. The UV lamps inactivate bacteria at rates proportional to the wastewater flow velocity, the effective volume of UV light penetration, the UV intensity within the reactor and the quality of the wastewater to be treated. Furthermore, the effectiveness of UV disinfection depends upon the care taken to control operations efficiently while reducing stoppages and maintaining performance.

Regular sampling and analysis of influent is essential in effectively controlling a UV disinfection system. Although there are no set rules determining the frequency or type of testing, it is generally accepted that the influent to the UV system should be analyzed for suspended solids, UV absorbance and coliform density. Furthermore, the flow rate, the number of lamps in operation and the coliform density in the effluent should be tested.

THE BANFF TOWNSITE UV SYSTEM

The BTSTF UV disinfection unit consists of two channels, each consisting of two horizontal modules with UV lamp banks placed parallel to the effluent flow. Combined effluent from the secondary clarifiers feeds into two channels of UV lamps. Downstream level control gates keep effluent levels high enough to submerge the UV lamps. A control panel monitors each channel with a UV intensity gauge registering the measurements of a sensor mounted in the channels. Adequate UV lamp intensity and turbulent flow design to a capacity of 25.5 ML/d inactivates coliform bacteria and thus reduces any chance of pathogenic bacteria discharge into the Bow River.

While the BTSTF allows manual or automatic operation, manual control may cut energy consumption in half during minimum flow periods (< 12.75 ML/d). The operator may either remove one channel from service by closing

The BTSTF's UV disinfection unit is monitored by a control panel. This control panel alerts the operator of any required cleaning and maintenance. Should a lamp burn out, the operator is alerted by a corresponding indicator light. Alternatively, if a bank of lamps ages or needs cleaning, daily UV intensity readings will signal that the operator's attention is required. Intensity meters indicate the relative intensities and their urgency for repair. For example:

1-3 = RED (poor)
3-8 = YELLOW (satisfactory)
8-10 = GREEN (good) (in mwatt/cm^2)

Fecal coliform bacteria kill is most efficient when the UV intensity meters are above 3 mwatt/cm^2, suspended solids are below 25 mg/L and average effluent flows are about 14 ML/d.

OPERATION

Since coming on-line in 1989, the BTSTF UV disinfection system has continued to undergo experimentation, analysis and modification. The BTSTF operators continue to perfect a method of maintaining the UV disinfection system efficiently and economically. Subsequently, a few of the many advantages that the UV disinfection system enjoys include; the advantages of low maintenance, no chlorine problems and a higher bacteria kill rate than many other non-UV systems.

The BTSTF UV disinfection operations continue to evolve through modifications improving the system's efficiency while reducing operating costs. Initially, cleaning was performed according to intensity readings of less than 1.0 w.s/m^2. The first cleaning took place only eight weeks after startup. A cleaning would consist of washing the lamps with low pressure water to remove lose debris, applying a scale remover, manually wiping each lamp and finally, applying a coating of Trojan 5 Scale Inhibitor. After a few cleanings the operators decided that the frequency of cleaning was too costly and time consuming. Subsequently, the operators changed three aspects of operation. First, they reduced their frequency of cleaning from once every three weeks to approximately once every three months by switching from UV lamp intensity readings to bacteria analysis. Second, intensity meters were re-calibrated adjusted upwards in order to allow a larger monitoring range between cleanings and thus a better indication of cleaning necessity. Finally, use of the Trojan 5 scale inhibitor was discontinued since there was no noticeable gain in inhibiting scale.

Presently, the operator need only worry about periodic cleaning and replacement of UV lamps whenever a count greater than or equal to 30 fecal coliform per 100 mL is detected. On average this interval of cleaning varies between 1600 and 3300 hours of usage. Alternating the usage of UV banks extends the interval of cleaning to once every spring and fall.

A cleaning begins by closing upstream isolation gates and thereby shutting down two banks in each channel. The operator cleans a 4 ft. long, 9 rack bank of 72 lamps by individually removing a rack of 8 lamps (15 lbs) each for a quick wash of low pressure water. Following a wash away of any loose debris, the operator sprays a scale remover (Lime-Away) on the bank of lamps before replacing any broken or burnt-out bulbs. The cleaning of a bank of UV lamps returns the UV intensity to a green (8-10 mwatt/cm^2) reading.

4.0 COMPOSTING

INTRODUCTION

Sludge composting is neither a new nor an innovative process, yet its use has only gained prominence in the last twenty years. The composting of municipal wastewater sludge has, in part, become more popular by society's shifting emphasis towards reducing wastes. Increasingly apparent are the benefits of composted sludge to soils. The improved physical properties include increased water content and retention for sandy soils, increased aeration and permeability for clay soils, greater root depth, increased microbial population and decreased surface crusting. Moreover, there are several advantages to using the sludge composting process.

Sludge composting is the aerobic thermophilic decomposition of organic constituents to a relatively stable humus-like material. The fundamental composting process has three key elements. First, the addition of a bulking agent to dewatered sludge creates a situation where the proper amounts of porosity, moisture and carbon contribute to the mixture's structural stability and the ability to create a self-sustaining decomposition reaction. Second, an adequate temperature range (55 to 65 C) induced by microbial activity destroys pathogenic organisms. Third, extended storage and regular mixing reduce moisture levels and aid in composting.

THE BANFF COMPOSTING PROCESS

The BTSTF composting process starts with dewatered sludge from 4 Autoheated Thermophilic Aerobic Digesters (ATAD) piled on an asphalt storage pad. The dewatered sludge is mixed in a 2:1:1 ratio with old composted sludge and woodchips to obtain a compost mixture high in nitrogen and carbon content. The woodchips serve as the bulking component of the approximately 40% solids moisture content. Subsequently, the compost mixture is piled in approximately 4 ft. high triangular windrows and mixed by a "Wildcat" Flail on a regular basis during the summer months. Approximately 80 to 90 m ^ 3/day of dewatered sludge is windrowed in the summer or stored in the winter. Nutrient analysis indicates concentrations of nitrate, potassium and sulfur content are ideal for park use.

OPERATION

The BTSTF compost process occurs only in the summer months due to Banff's cold winter weather conditions. Operators mix woodchips and old compost into dewatered sludge 4 to 5 days/week. Following this, the operators store the compost mixture in triangular windrows for an extended period of time. Provided that rain does not inhibit mixing, windrow mixing takes approximately 4 to 7 hrs/day every two weeks. Moreover, the operator does not have to allocate very much time or energy to realize the rewards of composting.

ADVANCES IN WASTEWATER SAMPLING

Chuck Baker
American Sigma, Inc.

INTRODUCTION

Determining and improving the quality of any body of water requires that pollutants be identified and monitored. The first step in determining the quality of a surface water is to repeatedly gather samples considered to be representative. The samples are then analyzed by a laboratory to determine water quality.

The collection of truly representative samples is critical. The subsequent laboratory analysis results in conclusions and actions that depend upon the validity of the sample. Collection of a representative sample is difficult because of flow variation and actual variation in the sample itself. Well designed and implemented sampling programs are vital to the pollution control process. The design of sampling equipment has to take into consideration all of the following parameters.

WHERE SAMPLES ARE TAKEN

Industry

Samples are taken to monitor the effect of various processes on the water used by industry. In addition to waste water monitoring, process and cooling waters may be monitored. Industry may also be required to monitor stormwater runoff from specific areas within their site

Municipalities

Samples are taken to monitor industrial effluent, stormwater runoff, waste treatment plant influent, the plant process and final discharge. (See Figure 1.)

Provincial and Federal

Samples maybe taken to monitor all of the above plus natural and man-made bodies of water.

SAMPLING METHODOLOGY

Sampling is necessary to gather information about the character of wastewater discharges and their impact on receiving waters. Many pollution abatement decisions are based on the analyses of the samples that are collected. These decisions will be based upon incorrect information if sampling is not carried out properly. Good sampling technique must be followed in collecting, preserving and transporting samples if reliable results are to be achieved. The following are considerations for any sampling program.

Fig. I - Typical Sampler Locations

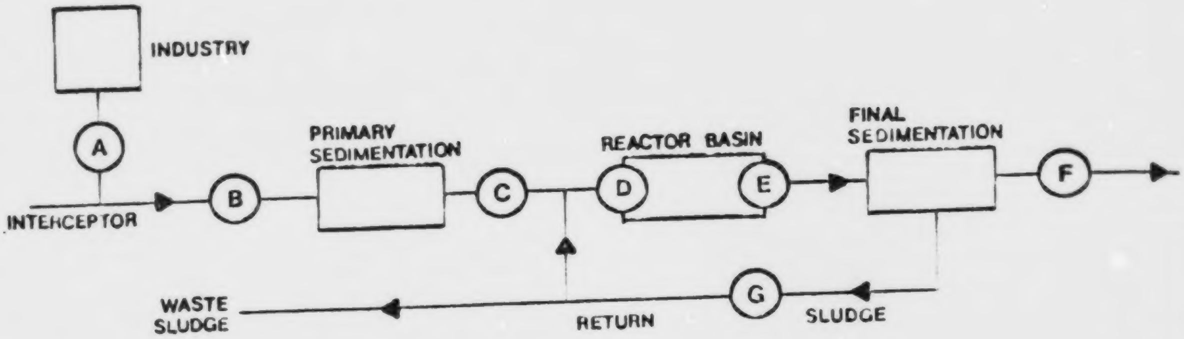


TABLE I. Sampling and Use of Data

Location	Information Obtained	Use of Data
A, B, C, D, E, F, G,	The shape of the oxygen consumption rate curve will indicate bioactivity of sample.	To determine treatability, toxicity, kinetic variables, and general metabolic conditions of the wastewater.
A	Strength, treatability or toxicity of industrial wastes.	To spot shock loads and to measure strength for sewer user charge.
B, C	Indicator of changes in plant loading	To operate plant using F:M ratio.
D	The demand for oxygen at the inlet to aeration tank.	Control aerators by matching oxygen supply with oxygen demand.
E	Oxygen consumption rate at outlet will predict BOD in final effluent	To estimate BOD in final effluent and control rate of return sludge.
F	Oxygen consumption rate of effluent.	To measure stream loading.
G	An indication of viable microorganisms in return sludge (M in F:M ratio).	To operate plant using F:M ratio.

Site Selection

Does the site provide the information desired?

Is the flow at the location known? If not, can it be reliably estimated or measured?

The site should be accessible. In sampling sewer flow, manholes located in busy traffic areas should be avoided. Manhole sites with a history of surcharging should also be avoided.

The site should be far enough downstream from tributary inflow to ensure full mixing of the tributary with the main stream.

The site should have turbulent flow. Suspended solids and floating oil and grease can create problems in gathering a sample. Materials that tend to stratify may bias samples depending upon the position of the sample intake. Location immediately downstream from a drop or hydraulic jump such as a flume generally provide the turbulence necessary for representative sample collection.

Type of sample to be collected

Is the waste composition variable? If no, a composite sample provide reliable date while minimizing analytical expense. If the composition is variable a discrete or sequential sample is required.

Is the flow variable? If flow is not variable a timed based sample will provide reliable data. If flow is variable a flow paced sample program is required.

Frequency of Sample Collection

Generally, the greater the frequency that samples are collected the more accurate the result. With highly variable conditions, samples should be collected as frequently as every five minutes to once hourly. The important consideration here again is the variability of the water composition.

Quantity of Sample

The sample volume required is dependent upon the the number and nature of the parameters that the laboratory is interested in. The methods and instrumentation used by the laboratory may also affect the volume required for analyses. Most test parameters require from 100ml to 500ml of sample. The volume of sample taken should be enough to perform all analyses, and to provide for errors, spillage and sample splitting. Generally a minimum volume of 500 ml is required for discrete samplers and 8 liters for composite samples.

Positioning the sample intake

The position of the sample intake within the flow cross section will have an effect on the outcome of that sample. Velocities vary within the cross section with higher velocities close to the surface and lower velocities near the bottom (average velocity in the vertical is about 0.6 depth. Lower velocities near the channel sides may create solid deposits while higher velocities in mid-stream better maintain solids in suspension. Also,larger solids tend to move to the outside of a curving flow. The effect of curvature on flow may continue for a considerable distance downstream affecting the distribution of suspended matter. The concentration of suspended solids increase with depth. The channel bottom may be completely covered by heavier solids while lighter materials such as oil and grease float on the surface. Consequently, a sample line placed near the bottom of the stream will collect disproportionate solids. When the intake is placed below the surface, floating material may be missed. However, a sample line placed in a straight turbulent section of the channel will produce representative results.

Sample intake considerations

For representative collection, the sample intake velocity should approximate the flowing velocity of the source being sampled. The operator should have an idea of the sizes of solids in the waste stream. The solid material obviously should not be larger than the diameter of the sample intake. If so and particularly if the material is stringy or fibrous, a strainer should be utilized. It is always desirable that the sample intake offer a minimal obstruction to the flow to prevent a build-up of debris and to prevent blockage. The sample intake line should be positioned downhill and free of loops to allow the purge to clear the full length of tubing. Generally, placing the sampler as close to the source as possible is desirable. The sample intake should point into the direction that flow is coming from. Other intake positions cause the the sample to undergo a change of direction causing segregation and a loss of solids.

Sample preservation

Many water and waste samples are unstable. In situations where the interval between sample collection and analysis is long enough to produce changes in either the concentration or physical state of the constituent, preservation is recommended. Preservation methods, consisting usually of pH control, chemical addition, and/or refrigeration, are intended to retard the chemical and biological changes, retard hydrolysis of chemical compounds and complexes and reduce volatility of constituents. The best assurance against sample deterioration is prompt analysis. However, with 24 hour or longer sample programs, a portion of the sample is stored for the entire program period and some type of preservation is required.

Sample types

The grab sample represents the conditions at the time the sample is collected. Grab samplers are usually not collected often enough at a given site to be statistically valid, and therefore, should not be used as a given basis for decision.

The composite sample is composed of a series of smaller samples collected in a single container and blended for analysis. Blending averages variations in waste composition and minimizes analytical effort and expense. Composite samples provide reliable data if the waste characteristics remain constant.

Discrete samples are composed of individual samples collected in individual sample bottles. Each bottle represents one segment of time. Analyses are done individually as well. Discrete samples are informative when sample composition is variable.

Timed cycle samples are collected in equal volumes at regular time intervals. Samples collected in this manner provide a representative sample only when flow volume is constant.

Flow proportional samples are collected in relation to the volume of flow during the sampling period. During variable flow conditions it is necessary to represent greater flows with proportionally greater sample than during periods of low flow. Samples can be proprietor two ways, by collecting equal volume samples over unequal periods of time or by collecting unequal volumes over equal periods of time. Samples may be proportioned either manually when flow records are available or by an automatic flowmeter.

Samples are more representative when collected automatically because each aliquot is collected identically. Therefore automatic samplers "standardize the error" and assure repeatability in the sampling program.

TYPES OF SAMPLERS

Prior to 1987 two types of automatic samplers where available.

The vacuum sampler offered high intake line velocities and repeatable sample volumes. It did not provide continuous flow from the sample source to the sample container. The metering chamber could not be rinsed and was a source of cross pollution.

The pre 1987 peristaltic pump sampler had lower intake velocities and suffered from poor volume repeatability caused by lift changes. The peristaltic pump sampler did provide continuous flow from the sample source to the sampler container.

SAMPLE PROGRAMS

Traditional sample programs permitted the operator to select from time based or flow based programs. Sample volume had to be calibrated on site in order to match lift conditions.

DEVELOPMENTS

In 1987 a high speed peristaltic pump sampler was introduced that used a liquid presence sensor to determine the presence of liquid at the pump. This sensor, combined with a microprocessor not only overcame the volume repeatability problem but provided intake line rinses and knowledge of missed samples. The program also permitted users to "try again" if a sample was missed. The high speed peristaltic pump offered velocities of 60 cm/s at lifts above 5 meters.

Programming capabilities were extended to include Samples/Bottle and Bottles/Sample in multiple bottle (2,4,8 and 24) configurations. Operators could now place the same sample in more than one bottle or more than one sample in a number of bottles. This feature allowed the use of more than one preservative for sample preservation. The addition of a timed delay feature permitted operators to place equipment sometime prior to the start of the program. Additional features in today's sample programs include Timed Override, a program that samples on the basis of flow but takes a sample on a timed basis if none is taken due to low flow. In addition, most samplers can now be started by a remote dry contact signal. A feature that extends sampler start parameters to level, pH, conductivity, etc.

By 1990 the general trend away from timed paced samples to flow paced samples had created a large market for portable open channel flow meters to pace samplers. Permanently installed samplers were easily paced by on site flow metering equipment. Field personnel assigned to monitoring off site discharges had to cope with heavy samplers containing as much as 24 kilograms of sample AND portable flow monitoring equipment which weighed as much as 10 kilograms.

In mid 1990 a portable sampler with integral flow monitoring capability was introduced. The use of vacuum formed materials significantly reduced sampler weight and the addition of the flow monitoring feature reduced combined sampler/flow meter weight by 11 kilograms. Both flow and sample data are maintained in an internal datalogger and may be transferred directly to a PC or downloaded to a DTU for transfer to either a PC or printer for a hard copy report.

Increased monitoring of industrial effluent has created the need to trigger industrial sampling by either pH or ORP (Redox) deviation. pH and ORP monitoring/triggering became a part of the sampling world with the early 1991 introduction of a sampler with an integral ph/ORP meter. Users may select upper and lower limits of either pH or ORP that are acceptable and trigger sampling on the basis of deviation. pH/ORP and sampling data can be transferred to a PC program for retention or a printer for hard copy report.

By summer of 1991 concerns over the polluting effect of stormwater runoff created the need for a sampler with multiple program start parameters and multichannel data logging capability. The typical stormwater sampler can be started from either rainfall, flow or level. The stormwater program takes a large sample when started, waits 30 minutes for the sewer to flush and then takes samples on either a flow or timed basis. The dual channel datalogger records flow, rainfall and samples taken.

CONCLUSION

Most of the recent developments in sampler technology have been dictated by the marketplace and relate, for the most part, to programming changes. The mechanical components of today's samplers appears to have reached an optimum and little change in their configuration is expected. Continued developments may include sample velocity control and multichannel, multiparameter datalogging of the samples physical parameters.

A WATER SUPPLY STRATEGY FOR THE PEMBINA TRIANGLE

by

G. H. MACKAY, Winnipeg

I BACKGROUND

I.1 General

The Pembina Triangle, the western portion of the Red River Valley in Manitoba, (Figure 1), has a population of about 38,000 (1986); about 50% rural and 50% urban. The population is growing at a rate in excess of the average for Manitoba, particularly in the larger urban centres (about 9% during the first half of the 1980's). Further growth in population and concurrent economic activity is constrained by a lack of safe, secure supplies of water for municipal and on-farm use.

I.2 Socio-Economic Circumstances

The agricultural productive capacity of the region is supported by an excellent soil base and climate. Special crops (potatoes; sugar beets; beans; etc.) along with traditional small grains and livestock make up the agriculture production.

During the past few decades a strong industrial sector has developed. The 1989 total employment by all goods and service firms in the major urban centres was 7619. The manufacturing sector, which accounted for over 2500 jobs in 1989, grew rapidly during the 1980's; 10.1% per year (as measured by shipments) during 1982 to 1986 as compared to 5.5% for all of Manitoba. The growth in employment during this period was over 7% per year, compared to a decline of about 0.3% in Manitoba.

The strength of the manufacturing sector in the Pembina Triangle in contrast to rest of the Province can be attributed to a numbers of factors. An innovative, entrepreneurial group exists (both rural and urban) which associated with a skilled, motivated work force gives the region a significant competitive advantage which is poised to capture the opportunities resulting from the recent free trade arrangements between Canada and the United States.

The opportunities for continued economic growth are viewed by local businesses as being excellent. Forecasts of economic growth have been made by local manufacturers and businesses in the region, based on the past record of growth and their current expectations regarding markets and their ability to compete in the North America trade framework during the 1990's and beyond. The following is a summary of the current expectations of local bussinessmen:

1. Projections are for an annual increase in total sales of 5.8 percent for the period 1989 to 1999 (the 1986 to 1989 growth was 4.7 percent annually). The projected annual increase for manufacturing firms only, was 5.0 percent projected growth from 1989 to 1999.
2. Employment is estimated to increase by 3.6 percent annually for the 1989 to 1999 period (estimated annual growth from 1986 to 1989 was 5.8 percent). Projected annual employment growth for manufacturing firms only is 3.8 percent from 1989 to 1999.
3. Direct economic impacts associated with incremental economic growth in the Pembina Region's manufacturing sector have been estimated for the next 20 years as follows:

   Employment
   - 18,802 persons years for the Pembina Region
   - 27,953 person years for Manitoba (includes the Pembina Region)
   - 32,921 person years for Canada (include Manitoba)

   Payroll
   - $545.6 million for the Pembina Region
   - $756.2 million for Manitoba
   - $890.1 million for Canada

4. If the manufacturing sector continues to represent the major percentage of total industrial growth, then the total 1991-2010 incremental growth in payroll and employment (includes manufacturing and non-manufacturing) will be:

Employment
- 66,542 person years for th Pembina Region
- 75,693 person years for Manitoba
- 80,660 person years for Canada

Payroll
- $1,165.6 million for the Pembina Region
- $1,376.5 million for Manitoba
- $1,510.1 million for Canada

In summary the potential for continued economic growth in this region exists. However without the provision of, amongst other things, additional sources of water for municipal and on-farm use, the growth potential cannot be realized.

I.3 Regional Water Resources

Notwithstanding the opportunities for economic growth within the Pembina triangle, there are a number of constraints that may prevent the capture of these potential opportunities. The most significant constraint is the meager water endowment within the region. Useable groundwater resources are limited to the Winkler Aquifer and Miami Aquifer along with a few sand and gravel deposits along the base of the Pembina Escarpment. The sustainable yield of the Winkler Aquifer is about 300 ac-ft/year (in the short term, larger withdrawals may be feasible) while the firm yield of the Miami Aquifer has been roughly estimated to be 200 ac-ft/year. There is a large bedrock aquifer underlying the Region; however the water is very saline (up to 30,000 ppm) and hence not useful as a potable supply.

Surface water resources are limited to the small eastward flowing tributaries of the Red River rising on the Manitoba Escarpment; ie Dead Horse Creek; Tobacco Creek; the Boyne River, etc. These streams drain relatively small areas (total of about 400 sq. miles) and are very sensitive to drought. There is little, if any run-off in dry years. Hence fairly large, expensive storage reservoirs would have to be developed to assure an adequate sustainable supply from these watersheds.

I.4 Existing Water Development

The constraining impact to regional development in the Pembina Triangle of the limited water supplies has been recognized for years. Over the past one half century local water developments have been implemented to supply the urban centers, while farm dug-outs and water hauling have served the rural areas.

A dam on Dead Horse Creek was constructed in the early 1940's to serve as a source of supply for Morden. The dam has been raised several times since (the last increase in the early 1980's); however, topographical constraints limit further increases. The Stephenfield Dam an the Boyne River upstream from Carman was constructed in the late 1950's and raised in 1990. This development provides for the municipal requirements of the Town of Carman along with some agricultural uses. Physical limitations restrict further development at this site.

In the early 1960's arrangements were made to pipe water from the Pembina River at Neche, North Dakota, to provide a municipal source for Altona and environs. Requirements for municipal water exceeded the supply available from Neche in the late 1970's. Thus a regional supply system was developed in the early 1980's which treats water withdrawn from the Red River at Letellier and transfers it via a pipeline to Altona to supplement the supply from Neche. Rural residents along the 13 mile pipeline from Letellier to Altona make use of the water via a network of rural regional pipelines. Some 900 farmsteads are hooked up to this system.

In 1961 Winkler began to use the Winkler Aquifer as a source for its municipal system. However it was recognized that the availability of fresh water from the aguifer which contains some fresh water overlying a large reservoir of salty water, was limited. Recent studies have shown that the average annual recharge of fresh water to the aquifer is about 300 ac-ft while current withdrawals are in excess of 1000 ac-ft. Salt water intrusion into the fresh water at the top of the aquifer is taking place and will likely accelerate if current withdrawals continue. It is likely that as the intrusion of salty water continues the entire aquifer will be useless as a source for municipal supplies.

Communities in the Eastern portions of the Region withdraw water from the Red River. However, during dry years, such as have been experienced over much of the last decade, the flows in the Red River, which to a large extent are derived from storage reservoirs in the United States, have fallen to alarmingly low levels. The security of this supply is questionable and its utility is further questioned by accidental spills from upstream industrial facilities, which occur from time to time during the late fall and winter.

I.5 The Valley Water Task Force

The recent drought brought to everyone's attention the fragility of the existing sources of supplies and led to concerns that the current pace of economic growth was threatened and in some cases may have to decline due to inadequate water supplies.

The municipalities within the Pembina Triangle, both urban and rural, realized in the early summer of 1989 that the past piecemeal approaches to developing municipal supplies was not in the best interest of the Region and that a regional solution to a regional problem should be developed. Accordingly a Water Task Force, representing all municipal governments (15) within the area of concern (Figure 1), was established with a mandate to examine the potential future water requirements for municipal and domestic use and to review options to supply these long-term requirements, having regard to the current drought driven needs. The Task Force was directed to select a long-term supply strategy that was cost effective; environmentally begnin; and acceptable to all concerned.

The Task Force sought, and obtained financial and technical support from the two senior governments.

II POPULATION AND WATER REQUIREMENTS PROJECTIONS

II.1 General

In order to develop a water supply strategy, estimates of future populations and the amount of water required too meet the needs of the estimated population, along with the concurrent economic activity are required. The Task Force was well aware of the need to conserve water and has agreed that none of the projections should include the inclusion of "wet" or water use intensive industries. The Task Force recognized that projections into the future were at best guesses; however guessing had to be undertaken.

The Task Force also recognized that the longer the look into the future the better the chance the "guess" would be wrong. On the other hand a long term strategy was considered essential. Accordingly the Task Force agreed that projections to the year 2040 would be attempted.

II.2 Population Projections

The Task Force requested the Manitoba Department of Rural Development to undertake a population projection study for both the urban centres and the rural areas. As well, each member municipality was requested to make population projections.

The Following table provides current population data from Statistics Canada.

TABLE 1

Town of R.M.	Population		Trend
	1981	1986	%
R.M.Dufferin	2408	2500	+ 3.7
R.M.Thompson	1364	1293	- 5.2
R.M.Stanley	4257	4623	+ 3.3
R.M.Rhineland	4473	4321	- 3.4
R.M.Roland	974	1014	+ 4.1
R.M.Morris(all)	2971	2937	- 1.1
R.M.Montcalm(all)	1769	1706	- 3.6
Morden	4579	5005	+ 9.3
Winkler	5046	5926	+17.4
Carman	2408	2500	+ 3.7
Morris	1570	1613	+ 2.7
Plum Coulee	592	677	+14.4
Altona	2809	2958	+ 5.3
Gretna	545	503	- 7.7
Emerson	762	725	- 4.8
Total Rural	18216	18394	+ 1.0%
Total Urbal	18311	19906	+ 8.7%

With respect to the current situation respecting livestock populations, the following data were provided by Statistics Canada.

TABLE 2

R.M.	TOTAL CATTLE		TOTAL HOGS	
	1981	1986	1981	1986
Dufferin	4758	4493	30285	26754
Thompson	2609	1923	8041	7206
Stanley	6495	6112	15370	18485
Rhineland	1850	1845	27800	35650
Roland	416	306	4810	4910
Morris	588	493	26379	22278
Montcalm	611	574	8676	11665

Upon completion of the two population projection estimates, the Task Force examined the results and agreed to the following for the purpose of the development of a water supply strategy.

TABLE 3

TOWN	2040 POPULATION
Altona	18000
Carman	5000
Emerson	725
Gretna	503
Morden	13038
Morris	10000
Plum Coulee	2877
Winkler	33520
Total Urban	83663
Total Rural	25702

II.3 Water Use Projections

The projections of the per capita consumption were clouded by a number of issues. Increasing water rates will tend to suppress consumption; on the other hand the area has grown under a water short environment and hence reductions in the current trends will not likely change. The participating municipalities are committed to vigorous water conservation initiatives to ensure that wastage will be reduced as far as practical. It was concluded therefore, for the purpose of sizing and costing of the required infrastructure, water use trends during the dry early 1980's were to be applied to the projected population to arrive at water demands at 2040.

TABLE 4

TOWN	WATER NEEDS (MILLIONS OF LITRES)
Altona	2980.5
Carman	1130.0
Emerson	90.5
Gretna	66.8
Morden	2902.7
Morris	1540.0
Plum Coulee	347.8
Winkler	6050.7
Total Urban	15108.5
Total Rural	2345.8
Total Livestock	7426.0
Total	24881.0

The current water use within the Pembina triangle is about 4000 million litres per year. The estimated use in the 50th year, ie in 2040, represents an increase of about 20 000 million litres over current use.

III WATER SUPPLY OPTIONS

III.1 General

The water supply problems within the Pembina Triangle have been of concern to area residents and to both senior governments for many decades. As a result many studies have been completed on many options to provide secure supplies. Hence there was available to the Task Force a considerable amount of information respecting options that should be examined. After reviewing the data the Task Force concluded that five options, on combinations thereof, should be examined in detail. These are as follows:

1. Develop reservoirs on eastward flowing streams rising on the Pembina Escarpment (Local Sources).
2. Create a storage reservoir on the Pembina River near Kaleida and pump from this reservoir to the headwaters of Dead Horse Creek where it would be treated and distributed throughout the region (Pembina River Source).
3. Divert water from the Red River near Morris where it would be treated and distributed throughout the region (Red River Source).
4. Divert water from the Assiniboine River northeast of Treherne to the Upper Boyne River to the Stephenfield Reservoir where it would be treated and distributed throughout the region (Assiniboine River Source).
5. Withdraw water from an aquifer east of the Red River in the vicinity of Marchand from where it would be piped to Morris, treated as required, and distributed throughout the region (Groundwater Source)

The general layouts of the five options are shown on figure 2 to 6 inclusive.
Each of the five options was examined from several perceptives:

1. Cost (excluding treatment)
2. Reliability of supply
3. Potential Environmental Impacts
4. Public Acceptance

The following is a summary of the findings that were available after the preliminary review.

III.2 Local Sources

a) Description: Reservoirs would be constructed on; Dead Horse Creek (upstream from the existing Morden Reservoir); Thornhill Creek; Shannon Creek; and The Boyne River.
b) Estimated Cost: $40 million ±25% (excluding treatment)
c) Reliability and Supply: The reservoirs would be on small streams with small contributing drainage areas (except for the Boyne River site) sensitive to long term droughts. Multi-year storage would be required. The security of supply was deemed to be risky.
d) Potential Environmental Impacts: Expected to be relatively minor, relating mostly to impact on habitat in the wooded ravines that would to flooded.
e) Public Acceptance: no expected concerns.

III.3 Pembina River Source

a) Description: A dam near Kaleida would create a reservoir in the Pembina Valley of sufficient capacity to ensure adequate water during a prolonged drought. Water would be pumped from the reservoir about 200 ft vertically to the headwaters of Dead Horse Creek where it would be treated and distributed to the western and central portions of the region while the Red River would continue to supply the Red River Communities.
b) Estimated Cost: $25 million ±25% (excluding treatment)
c) Reliability of Supply: The Pembina River drainage basin upstream from the proposed dam site is drought sensitive and long periods of zero flow have been observed. It also has to be recognized that the Pembina River flows into North Dakota and hence is an International Waterway and as such is likely subject to apportionment of its runoff between Canada and the United States. The amount of water rising in Manitoba that would have to be passed to the United States has not yet been agreed to. The security of supply was considered somewhat risky.
d) Potential Environmental Impacts: The reservoir on the Pembina River would have to be of such a size that several hundred acres of valley land would be flooded. The wildlife habitat loss and the impact on recreation, along with the loss of productive arable land was considered to have servere impacts.
e) Public acceptance: The concept of storing water in the Pembina River for use within the southern portion of the Pembina Triangle has been considered for several decades; hence there is some public awareness of the proposal. While no significant concerns were anticipated from users of the new water supplies it was expected that considerable concerns could be expected from residents of Pembina Valley.

III.4 Red River Source

a) Description: Intake works on the Red River near Morris would withdraw water from the River where it would be treated and then piped north to the Carman Area and south to the Morden - Winkler area. Existing facilities at Letellier, up-graded as required, would service the Red River communities along with the Altona-Grenta area.

b) Estimated Cost: $19 million ±25% (excluding treatment)

c) Reliability of Supply: Flows in the Red River are regulated by storage reservoirs in the United States. Low flows are augmentated by releases from these reservoirs. There are no formal arrangements with the United States respecting reservoir operations to ensure low flow augmentation at the International Boundary. During the drought of the 1930's (prior to the construction of the U.S. storage works) the flow in the Red River at Emerson dropped to very low values (less than 1 cubic foot per second during the winter of 1936-37). During the drought of the late 1980's the flows also dropped to levels that were cause for concern to users of the River in Manitoba. The security of supply, without some storage in Manitoba, either in the River Channel or in reservoirs adjacent to the River, is considered risky.

d) Potential Environmental Impact: The intake works at the Red River might require a low weir in the River to ensure an adequate depth of water at the pump intake. Concerns respecting fish movement could likely be mitigated by a fish-passage device. However the impact on riverine habitat could result in potentially significant adverse impacts.

e) Public Acceptance: No significant concerns expected.

III.5 Assiniboine River Source

a) Description: Intake works on the Assiniboine River would draw water from Assiniboine River and pump it about 180 ft. vertically to the top of the Assiniboine River Valley from where a short channel would discharge into the headwaters of the Boyne River. The diverted flow would be stored in the existing Stephenfield Reservoir. A regional water treatment plant at the reservoir would provided treated water for a pipeline distribution network east to Carman and south and east to supply the entire region.

b) Cost Estimate: $16 million +/-25%. (excluding treatment)

c) Security of Supply: Flows on the Assiniboine River are regulated by the Shellmouth Reservoir. Advice from the Manitoba Water Resources Branch was that a withdrawal of 25 cubic feet per second (about 10% of the expected future low flow in Assiniboine River) could be accommodated under existing operating procedures for the Shellmouth reservoir. Hence the security of the supply is considered to be adequate.

d) Potential Environmental Impacts: The potential impacts of the physical works are considered to be minimal as are the impacts on the downstream aquatic environment.

e) Public Acceptance: No significant concerns expected.

III.6 Groundwater Source

a) Description: There is an aquifer near Marchand in which a well field could be developed to withdraw water which would be piped to near Morris. Any required treatment would be provided and the water distributed throughout the region.

b) Estimated Cost: $60 million ±25%. (excluding treatment)

c) Security of Supply: The substantial yield of the aquifer is not known. A $500,000, two year investigation, would be required to evaluate the long term aquifer yield. The security of supply is therefore not known.

d) Potential Environmental Concerns: None expected.

e) Public Acceptance: Concerns regarding the security of the supply were expected to cause concerns to other potential aquifer users.

IV DESIGN OF THE WATER SUPPLY STRATEGY

IV.1 General

Upon completion of the studies of the options outlined in Chapter III, an extensive public consultations were undertaken to ensure that input from all the residents of the Pembina Valley respecting both the water needs projections and the options that were examined. Meetings were held with all Municipal Councils. Four open house meetings were held throughout the Pembina Triangle at which over 400 people had the opportunity to examine the options considered and to express views on the options. The Public Consultation process is described in detail in Chapter V.

Subsequent to the Council meetings and the "open houses" the Task Force undertok a comprehensive evaluation of all matters it considered germane in the development of its strategy. The estimated cost, the security of supply, the potential environmental impacts along with socio-economic considerations, and public acceptance, were carefully examined and discussed with the Technical Advisory Committee.

IV.2 The Selected Option

The Task Force unanimously agreed on a preferred strategy to supply the identified municipal and on-farm water requirements. The preferred option includes withdrawal of up to 20 cubic feet per second of water from the Assiniboine and diverting it to the Boyne River and then to the Stephenfield reservoir, treating it there, then distributing it east to Carman and south to the Morden-Winkler area. Farmsteads and small communities in the western portion of the Pembina Triangle between the Stephenfeild Reservoir and Morden-Winkler would be serviced with treated water. A pipeline running easterly from Winkler would provide treated water to Plum Coulee.

The easterly portions of the Region would be serviced by water withdrawn from the Red River (up to 5 cubic feet per second) near Morris and near Letelier. The Town and R.M. of Morris, including Rosenort, would be supplied from a treatment plant in the Town of Morris. The existing works at Letellier would be expanded as required to provide treated water to Altona, Gretna, St. Jean and Emerson.

The Task Force noted the Red and Assiniboine systems could be interconnected in the future to enhance the security of the entire system during drought. Provision for such an inter-connection is not included at this time.

The continued use of existing infrastructure, including water treatment plants, for the remainder of their useful life, and the continued use of existing sources of supply are integral components of the selected option.

On aggresive long-term water conservation program throughout the entire region completed the strategy developed by the Task Force and adopted by the member Municipalities. Figure 7 displays the various elements of the water supply system proposed.

IV.3 The Diversion Works, Main Distribution System and Water Treatment

The following sections describe in detail the various physical components of the water supply.

IV.3(A) General

The main water supply pipelines were sized for 50 year average day flows or the projected 20 year peak day flow rate, whichever was the greater. The total cost of the main water supply pipelines was estimated to be $22,609,000 including a booster pumping station north of the town of Morden.

The type of pipe analyzed for the networking was PVC series 125 plastic pipe which is an economical smooth wall plastic pipe used to convey treated water in many water supply pipelines throughout the province of Manitoba. It must be emphasized that the supply pipelines sizes were designed for what is commonly described as a "trickle system" which means that during the peak days the pipeline pressures may be as low as 20 to 30 pounds per square inch due to increased head loss during peak day flow rates.

Normally during average days the pressure can be sustained at higher levels (50-70 psi). This means that consumers along the pipeline route must normally repressurize from pumphouse and watermain distribution systems (in the case of towns and villages) or from the concrete cistern and pump installations (in the case of rural farms). The pressure in the supply pipelines would be higher in the initial demand years but would quickly decrease as flow rates in the pipeline increase.

Additional pipeline (rural pipelines) would be required to be constructed from the main supply pipelines to the farm areas. These rural pipelines would range in size from 200 mm (8 inch) diameter to 75 mm (3 inch) diameter. The location, sizing and costs of these rural pipelines have not been included here.

All pipelines would be buried a minimum of 2.7 metres (8.5 feet) with additional 1.3 meter (4.5 feet) depth for pipelines crossing roadway embankments and railways. All paved roadways would be tunneled to prevent damage to concrete or asphalt surfaces. Railway crossings would also be tunnelled and metal encasement pipes installed. The crossing of major rivers, lakes or streams (continuous flow channels) would be accomplished by using high density butt fused polyethylene pipe complete with concrete weights.

All pipeline installations would include manual air release valves, air and vacuum chambers, flushing valve assemblies, and gate valves for pipeline operation and maintenance.

The existing method of water treatment in the urban centres is predominately lime soda ash solids contact, recarbonation, filtration and chlorination. This method of water treatment was adopted for the proposed system. The sizing of the proposed water treatment facilities at Morris and Red River Regional expansion at Letellier was based upon average day consumption for rural and livestock as well as peak day rates for the urban centres. The sizing of the water treatment facility at Stephenfield was based upon average day demands for the rural, livestock and urban demand projections. The Towns of Winkler, Morden and Carman were assumed to continue to operate their existing facilities to treat those volumes of water available on a firm basis from existing sources (i.e. Winkler Aquifer, Morden Reservoir and the Boyne River).

Water treatment plants are generally designed for a 20 year design projection period because the normal life span of many treatment components is approximately 20-25 years depending on the type of construction.

The capacity of the treatment plant at Stephenfield is 232 litres per second at an estimated cost of $9,820,000 including engineering and contingencies.

The capacity of the proposed regional water treatment facility at Morris for year 2010 is 49 litres per second and is estimated to cost $3,950,000 including engineering and contingencies. The capacity of the proposed water treatment expansion at the Red River Regional Water Treatment Plant at Letellier for year 2010 is 66 litres per second at an estimated cost of $4,400,000 including engineering and contingencies.

The total estimated cost of pipelines and water treatment facilities to supply projected water requirements to the Pembina Triangle study area is therefore $40,779,000 including engineering and contingencies.

IV.3(B) The Diversion Works From the Assiniboine to the Stephenfield Reservoir

Location of Works

Geological features were the major factors considered in selecting the location for the proposed diversion works. Siting the river intake, pump station and transfer pipeline was governed by the geology of the Assiniboine River Valley. Virtually the entire reach of the valley in the general project area is mantled by slump debris. Construction of a large diameter pipeline in such material would be very hazardous because of the potential for further slumping and resultant rupturing of the pipeline (which would likely release a large volume of water, further destabilizing the slope and possibly endangering the pump station). A reconnaissance geologic evaluation did, however, identify two possible "windows" through this slump debris; in W 15-9-9 W1 and NW 13-9-9 W1. The slump free area is wider at the former site and the river channel is also relatively stable at this location with no major shifts in position during the past 40 years. Therefore, 15-9-9 W1 was selected as the location for the intake works, pump station and transfer pipeline.

The route for the transfer canal was chosen, after consultation with local residents, to provide a suitable slope for the canal, to avoid hydro lines and farm yards as much as possible and to minimize the loss of agricultural land. The Manitoba Soil Survey Report for the area was consulted and it indicates that most of the selected canal route would be though relatively impermeable soils so that lining may be required for only a short reach.

Boyne River Capacity

The capacity of the Boyne River was examined to determine whether additional work would be required along it's channel to pass the diversion flows. The capacity was estimated for the reach downstream from the confluence of the diversion canal and the Boyne river to the south boundary of 26-7-8 W1, a distance of 17 km. The water surface profile for the proposed diversion flow was calculated which indicated that the flow of 20 cubic feet per second would be contained within the channel.

Winter Flows

While the diversion works would be designed to be capable of maintaining year-round flows, it is not entirely certain that winter flows can be maintained in both the transfer canal and the natural Boyne River channel, without additional works.

Recent experience with the Assiniboine River to La Salle River diversion suggests that heavy snow accumulations can cause submergence of the ice cover, resulting in localized ice buildups and flooding. Moreover, low flow simulations indicate that where the Boyne River channel is not well defined, water depths would be very shallow and water velocities very low. This could result in "freezing to the bottom" situations.

Because of these concerns, the impact on Stephenfield Reservoir levels of not pumping for three winter months (January, February and March) was examined. Reservoir simulations indicate that without winter pumping the storage available in the Stephenfeild Reservoir would be capable of meeting existing Boyne River and projected Pembina triangle over-winter demands for well over 20 years into the future without significant drawdown. This amount of time would therefore be available to develop ways and means of overcoming ice problems should they be a concern.

River Intake Works

The intake would consist of a river intake structure, 0.9m (36 inch) diameter concrete intake pipe and an intake canal. The intake would be situated partly below the existing river bed elevation to provide a sufficient depth for winter withdrawals, and would be screened to prevent fish etc. from being swept into the pump. Local deepening of the river and periodic maintenance would be necessary around the intake. A gate in the pipe would permit the dewatering of the intake canal for maintenance purposes.

The minimum river flow was estimated to be 5.7m3/s (200 c.f.s). This corresponds to an estimated water level of 275.85 m and depth of flow of about 0.35 m (1 ft). The bottom of the intake canal would be about 1.5 m (5 ft) below the estimated minimum river water level to provide a sufficient depth of flow for winter operations. The canal was designed with a 3 m (10 ft) bottom width, 4:1 side slopes up to the minimum river level and 5:1 side slopes above. It would be approximately 400 m (1300 ft) long, 7 m (23 ft) deep and have a 67 m (67 ft) top width. The low velocities on the canal would allow some of the suspended sediment to settle out of the river water and thus reduce wear on the pumps. However, the canal would require periodic maintenance and the suitable disposal of the dredged material.

Pump Station

The pump station would consist of a precast concrete intake pipe, a cast-in-place reinforced concrete substructure, a reinforced masonry and structural steel superstructure, and pumping units and associated mechanical and electrical works.

Due to the high sediment concentrations in the Assiniboine River at certain times of the year (generally spring) it would be prudent to reduce or curtail pumping operations at these times in an effort to reduce wear and possible damage to the pumps.

Transfer Pipeline

The 13 km (8 mi) long, 0.8 m (32 in.) diameter PVC to be buried at a depth of 2,5 m (8 ft) is proposed to convey water from the pump station to the transfer canal. The pipeline was sized to minimize surge pressure and friction losses while maintaining an economical pipe size. Riprap would be used at the pipe outlet to protect the canal from erosion.

Transfer Canal

The 13 km (8 mi) long transfer canal would convey the water south and east to the Boyne River. A triangular cross section with 3:1 side slopes was chosen to provide a sufficient flow depth to facilitate winter operation. The proposed canal would have one steel sheet piling and rock riprap drop structure and have 0.9 m (36 in.) diameter corrugated steel pipe culverts to cross existing roads.

No detailed soils information exists for the canal route however it was assumed that approximately 1.5 km (1 mile) of the route would require lining to reduce seepage. This was based on the Manitoba Soil Survey Report No. 7 and a cursory field inspection. The need for lining as well as optimizing the canal design in terms of the elevation and slope of the canal invert and the number and location of the drop structures would be verified prior to final design. Moreover, local land drainage needs would be considered and incorporated into the design at that time.

Some reaches of the canal would have an excess of excavated material; this could be wasted in either spoil dikes along the edge of the canal or used to build up the existing road adjacent to the canal.

IV.3(C) The Diversion Works on the Red River

Location of Works

Withdrawals of up to five cfs from the Red River (in 2040) are proposed at the locations of the proposed water treatment plants near Morris and Letellier. The water use projections indicate that in 2040 the withdrawal at Letellier will average an additional three cfs while at Morris an additional two cfs will be withdrawn from the river. Existing withdrawals at Emerson and St. Jean will cease since these two communities will be serviced from the facilities at Letellier.

Red River Water Supply

During the past few years the flow in the Red River at the International Boundary has receded to levels which have been given rise to concerns that the existing intake works at Emerson, Letellier, St. Jean and Morris would not be able to function properly. Further the low flows have raised the possibility that the river may not be considered as a firm source of supply for the proposed regional system. The discharge in the river is, during low flow periods, augmented by releases from storage reservoirs in North Dakota and Minnedosa. The two major storages are the Baldhill Reservoir on the Sheyenne River with about 40,000 acre-ft of useable live storage and the Red Lakes Reservoir on the Red Lake River with over 850,000 acre-ft of live storage.

The reservoirs are operated to meet uses and requirements in the United States. However, the reservoir releases for United State's purposes result in higher than natural flows at the Boundary during drought years. There are no formal arrangements with the United States authorities respecting the maintenance of minimum flows at the Boundary. However, the Manitoba Water Resources Branch has advised that for the purposes of determining the security of supply for the Pembina Triangle Water Supply System, a minimum mean monthly flow of 65 cfs could be considered. During recent years the mean monthly flow has been as low as 172 cfs during January of 1990. During the protracted drought of the 1930's, the mean monthly flow dropped to 1.2 cfs in 1937. There were no major storage reservoirs in the United States during this period and hence these low flows were natural flows.

The diversion works on the Red River at Morris are proposed to be similar to those currently being used. They are proposed to consist of a river intake at or near the existing intake, with adequate protection to prevent debris and fish to be diverted into the pump works to be located in the wet well on the river bank.

From the wet well the water will move via a transfer line to the treatment plant. No significant change in the existing arrangements of the diversion and pumping works at Morris are proposed.

At Letellier the diversion works will be an expansion of existing system whereby water is pumped from the river to either the treatment plant or the raw water storage in the oxbow on Plum Coulee adjacent to the Red River.

Emergency Storage

In addition to concerns respecting the security of supply in the Red River, there are also concerns regarding accidental spills into the river from industrial plants upstream. Such spills have occurred from time to time in the recent past. There is an arrangement under the auspices of the International Joint Commission whereby the quality of the river is carefully monitored at the Boundary and an alert system is in place to warn users of the existence of poor quality water in the river at Emerson. However, there are no arrangements whereby minimum water quality standards are guaranteed.

In order to protect the system from a "slug" of poor quality water, emergency storage is proposed for both Morris and Letellier. After due consideration of available data, it has been concluded that two weeks storage would be sufficient.

In the case of Letellier, the existing oxbow storage will suffice until after 2000 at which time some additional excavation may be required. The two weeks of emergency storage will also reduce to some small degree the risk the implications to the security of the system in the event that flows in the river decline below the 65 cfs for short periods of time.

In the case of Morris, it is proposed to provide initially for 14 days storage at the level of withdrawal estimated for 2010 at which time the need for additional storage capacity will be reviewed. It is proposed to provide 38 acre-ft of storage, plus an allowance for up to three feet for ice, in a dugout to be constructed at a suitable site near the proposed treatment plant. For purposes of estimating the cost and land required it has been assumed that the 38 acre-ft of storage will be provided below ground while the provision for ice (about 15 acre-ft) will be stored above ground, contained by low dikes constructed from the material excavated from the dugout. Such a configuration will require about 20 acres of land and cost about $100,000. It is entirely possible that during final design a balance between excavation and dike requirement will result in considerable savings (a recent storage dugout at Grandview is constructed on this principle). The pumping facilities at the intake works will have provision to divert water either to the dugout (probably during the latter part of the spring runoff when the water quality is good), or to the treatment plant.

IV.3(D) The Pipeline Network

The study area would be serviced from three main regional water supply pipelines.
The first pipeline would commence at the regional water treatment plant proposed adjacent to the Stephenfield Reservoir. The pipeline from this plant would be 750 mm (30 inch) diameter and would continue on a south easterly direction across country for approximately 19 kilometers. A 15 meter (50 feet) width of pipeline easement would be purchased to facilitate construction. The remainder of the 750 mm pipeline to Morden would continue 17 kilometers in a southerly direction within the existing municipal road allowance located 1.6 kilometers (1 mile) west of provincial road 432 to a junction point approximately 1 km north of the Town of Morden Water Treatment Plant. The last 1 kilometer from the junction to the Town of Morden Water treatment Plant would be 450 mm (18 inch) diameter.

A 450 mm (18 inch) diameter pipeline would be constructed from the town of Morden 14 kilometers east to the Town of Winkler along provincial trunk highway (PTH) No. 3. A 200 mm (8 inch) diameter pipeline from Winkler to Plum Coulee completes this component of the west proposed system.

The pipeline to the Town of Carman, would consist of a 300 mm (12 inch) diameter pipeline for a distance of 20 kilometers east of the proposed regional water treatment plant site at Stephenfield. The pipeline would be located north of the Boyne River and would require two river crossings during construction.

The second proposed regional water pipeline would commence from a proposed water treatment plant site at the Red River to be located near the Town of Morris. A 300 mm (12 inch) diameter pipeline would be installed from this location 18 kilometers west along provincial trunk highway No.23 to the community of Lowe Farm. This pipeline would serve the Rural Municipality of Morris including the village of Rosenort.

The third regional water pipeline would begin from the existing regional water treatment plant in the Village of Letellier. The projected 50 year population for this area requires that a 400 mm (16 inch) diameter pipeline be constructed 21 kilometers to the Town of Altona. This pipeline would be installed approximately along the same route as the existing 300 mm (12 inch) diameter pipeline installed in 1985.

An 18 kilometer long 150 mm (6 inch) diameter pipeline would be constructed from the Red River Regional Water Treatment Plant to the Town of Emerson and a 16.5 kilometer 150 mm (6 inch) diameter pipeline would be constructed to the Village of St. Jean Baptiste. Those pipelines eliminate the need to replace existing water treatment plants which are near the end of their useful life.

The following table indicates the estimated cost for the various segments of the distribution system. the cost estimates are shown in 1990 dollars and include provision for engineering at 15% and contingencies at 15%.

TABLE 5

Segment	Length Meters	Size mm	Estimated Cost $1,000
Stephenfield Regional Water Treatment Plant To Junction PTH. No 3 near Morden	38,700	750	11,994
Junction PTH No. 3 to Morden	1,200	450	230
Morden to Winkler	12,000	450	1,690
Winkler to Plum Coulee	2,500	200	834
Stephenfield Regional Water Treatment Plant to Carman	19,500	300	1,658
Regional Water Treatment Plant near Morris to Lowe Farm	16,500	300	1,474
Morris To Rosenort	13,000	200	789
Letellier to Altona (required in 2000)	20,700	400	2,349
Letellier to Emerson	18,000	150	696
Letellier to St. Jean	15,600	150	630
Booster Station			265
Total Distribution System			22,609

IV.4 Waste Water Considerations

The urban, rural, and livestock water demands within the study area will generate wastewater effluent which will require treatment prior to release into the environment.

The present wastewater treatment facilities in the study area servicing the urban population consist predominantly of flocutative wastewater stabilization ponds(lagoons) which store sewage during the winter period from November 30 to May 15. This method of sewage treatment is anticipated to continue to be implemented until more rigid regulations dictate a change in treatment processes. The construction of sewage treatment plants in the future may necessitated as a result of restrictions being imposed.

Farmsteads presently utilize the septic tank and field method of treating domestic waste. Some residents may use holding tanks which are frequently pumped and wastes hauled to a wastewater stabilization pond or approved dumping site. The expansion or rural water use and consequently the wastewater volumes would result in more numerous septic tank/holding tank installations but it is anticipated that the overall strategy of treatment of rural domestic effluent wastes would not change.

Livestock wastes in the area consist of waste generated from cattle, hogs, and poultry. Many large hog operations presently consist of building enclosures which collect hog wastes through a slotted floor system. These wastes are then transferred to a wastewater pond utilizing anaerobic and aerobic wastewater treatment which eventually results in the land application of the animal wastes. This method of waste treatment is anticipated to continue in the future.

Cattle wastes generated in cattle holding areas are collected and utilized as manure for agricultural land application. As stock increase it is anticipated that there would be an increase in the intensity of manure application. Similarly, wastes generated from poultry operations would continue to be land applied in the future as is presently done.

V. COST ESTIMATE AND CONSTRUCTION SCHEDULE

V.I General

Estimated costs for each of the various elements were prepared by the several provincial and federal agencies involved in the design of the works. Allowance has been made for contingency factors to reflect the uncertainties associated with the designs and an estimate of the cost of engineering is also included. The estimated costs of interest during construction is included; however the capitalized value of operation and maintenance is not included. These costs, along with the costs of borrowing capital funds, will have to be included in the considerations leading to setting a unit price for treated water to the consumer.

A proposed construction schedule was prepared to illustrate a possible scenario for the timing of the completion of the various works and an enhanced cash flow.

V.2 Cost Estimates

The total estimated cost to provide a safe secure supply of potable water to all the residents within the study area is shown in the following table.

System Component	Estimated Cost Millions Dollars (1990 dollars)
Major Distribution System	22.6
Water Treatment Plants (including intakes)	18.7
Assiniboine-Boyne Diversion Works	3.3
Red River Emergency Storage	0.2
Total Capital Cost of Major Infrastructure	44.8
On-Farm Water Supply Works	19
Total Cost	63.8
SAY	65

It should be noted the total cost of $65 million is stated in 1990 dollars. The funds would likely be expended over a 10-year period. The annual capital expenditures over this period have not been discounted nor has consideration been given to annual inflation.

Annual operating charges would vary with the amount of water used. It is estimated that the annual operating and maintenance costs of these new facilities, including pumping and treatment, would be about $1.3 million in year one and about 2.8 million in year 20 (1990 dollars).

Note: Provisions of the proposed federal Goods and Services Tax have not been included in any of the cost estimates shown.

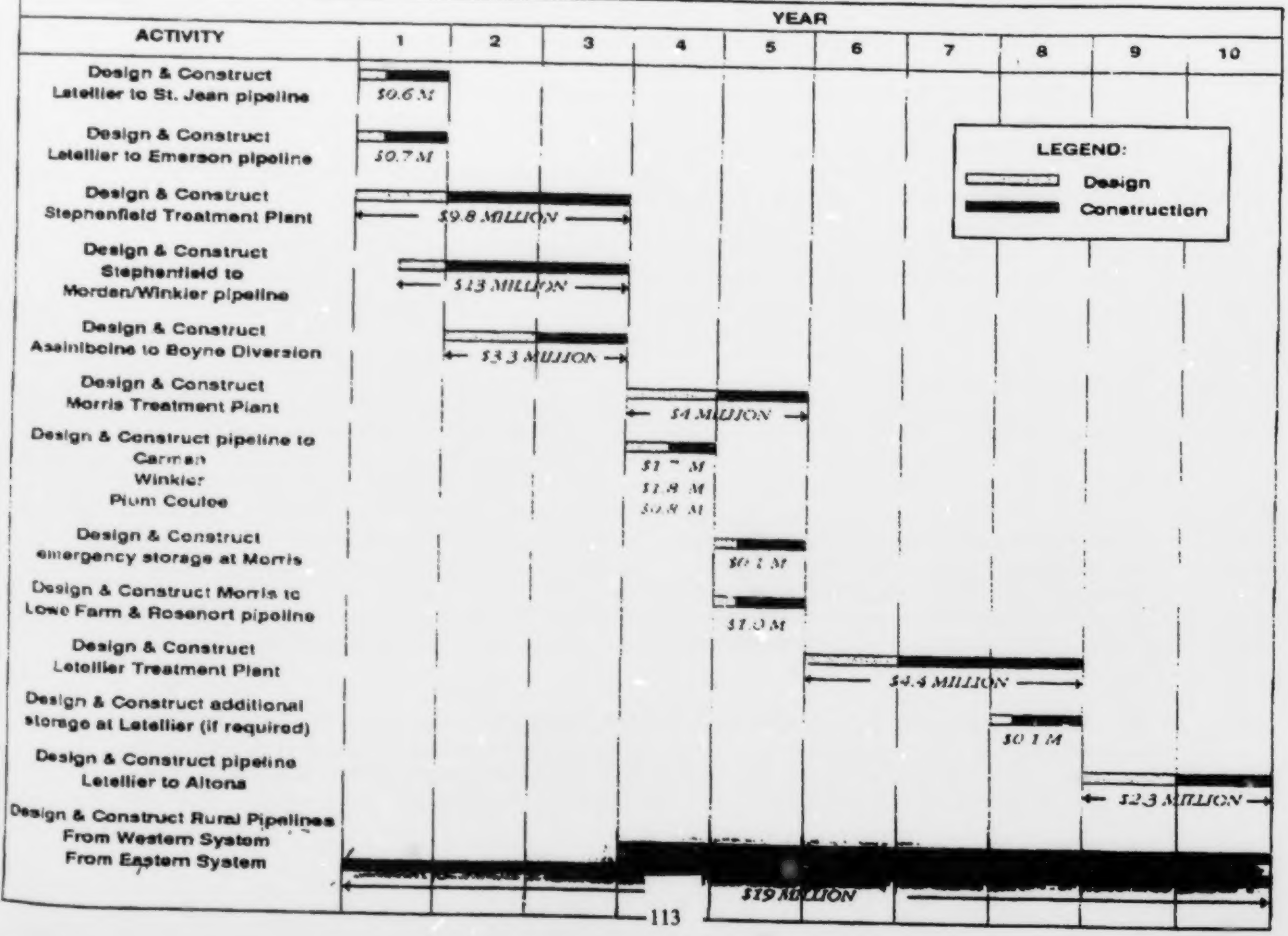
V.3 Construction Schedule

A notional construction schedule was developed based on a number of considerations as follows:

1. The implementation period for the first phase, that is to and make treated water available to all users within the project area, was considered to be 10 years. Such a period would allow a reasonable construction period and meet urgent needs. The second phase would involve upgrading existing facilities to ensure their continued beneficial use.
2. Those elements that are required to meet urgent needs are scheduled early in the decade. For example, the existing intake works at Emerson are in disrepair, therefore, a pipeline from the regional treatment plant at Letellier, where there is available capacity, to Emerson is proposed for year one.
3. System elements will be "phased in" as other components are completed. For example, there is an urgent need at Morden to "firm up" the supply, however, there is no point in completing the pipeline from the treatment plant at Stephenfield to Morden unless the treatment plant is operational. Further, there is no need to complete the diversion from the Assiniboine to the Boyne until the Stephenfield treatment plant is operational.
4. The implementation schedule must recognize the need to "levelize" the cash flow to an extent possible within the constraints of needs for water. In addition, the schedule has to recognize the ability of local contractors to have the opportunity to work on the projects. For example, two major treatment plants being constructed simultaneously could tax the capability of local contractors and the potential opportunity for lower prices could be lost.

The following chart is a notional implementation schedule based on the above principle. It is not "cast in stone" but is displayed to indicate a likely undertaking of construction activities.

Pembina Valley Water Co-op — Phase 1 Construction Schedule



VI PUBLIC CONSULTATION

VI.1 General

The Task Force recognized early in its activities that an effective ongoing public consultation process was essential to the success of the development of an acceptable long term water supply strategy. Accordingly arrangements were developed whereby all decisions were made after in-depth consultations with the residents of the Pembina Triangle. Information meetings with all the participating municipalities were held to describe the projected water requirements and the options that were being considered to ensure that these long term requirements would be met. These information meetings were followed by public information meetings throughout the region at which time public comment was solicited. In addition, media was invited to the meetings of the Task Force. In order to ensure that the potentially impacted interests outside of the region were aware of the activities of the Task Force, numerous information sessions were held in both the Assiniboine and Red River Watersheds.

VI.2 Public Meetings

When the task Force had identified the water supply options available to meet the agreed-to long term water demands, and the techniclal aspects of the options studied, the Task Force sponsored four public meetings to examine the options being considered and to express opinions and to ask questions. Memebers of the Technical Advisory Committee were present at these "open houses" as were members of the Task Force. Over 400 persons attended the meetings and useful dialogue took place. Opionions were expressed and taken note of by the Task Force.

VI.3 Meetings Outside of the Pembina Triangle

In order to consult interests that might have concerns about such matters as withdrawl of water from the Red and Assiniboine Rivers for municipal use in the Pembina Triangle and the possible impact on the operation of exisitng water developments works, the Task Force arranged to meet and discuss the studies and findings with several municipal councils. Informational meetings were held with: the R.M.'s of South Norfolk; Shellmouth; Russell; Shell River; Portage la Prairie; the towns of Roblin; Russell; the cities of Portage la Prairie, Selkirk and Winnipeg. The task Force also met with the Band Council of the Rouseau River Indian Reserve to discuss the proposed project.

VI.4 Public Consultation Results

The Task Force has concluded that its aggressive public communication strategy has ensured that all of its decisions were made with significant input by the residents of the area. In addition, the meetings with officals from jurisdictions outside of the area ensured that everyone who needs to know the details of the work and conclusions of the Task Force, does.

VII WATER CONSERVATION

VII.1 General

The Task Force recognizes that a water conservation initiative for the region is required as an integral component of a long term water supply strategy. The management of water demands has to be considered as important as the management of water supply. The region has devloped in a water scrace environment and the careful, prudent, use of water has been necessary. The fact that water users in the Pembina Triangle recognize that water is a scarce resource and must be used carefully can be demonstrated by comparing per capita consumption of similar urban communities within the region to those outside the region. Three centres, one within the region and two outside, none of which have an industrial base, were compared.

Nine years of data beginning in 1981/82 and ending in 1989/90 was commpared. The centre within the study area had the most expensive water and a nine year average water usage of 23 gallons per person per day. A centre outside the study area which had a water rate less than one quarter of the rate being paid in the study area, had an average per capita consumption of 54 gallons per person per day. The third centre which paid 12% of the per unit price charged by the study area, centre, had a per capita consumption of 70 gallons. While it is not possilbe to eliminate all the variables and make definite statements, it does appear safe to state that there is a relationship between the per unit cost of water and consumption and that the study areas' adoption of conservation has not been restricted to farmers who haul water. It also supports the user pay concept for water pricing as an effective management strategy.

The Task Force recognizes that if an additional commitment of the province's available water resources is to be made to the study area, it is essential that the water be conserved and utilized in a conscientious and prudent manner.

VII.2 Regional Survey of Industrial Users in the Study Area

As part of the economic survey conducted in the industrial, service and public sectors of the study area, an attempt was made to evaluate the attitude of business toward water shortages and conservation techniques by means of a questionaire. The preamble to the question was:

Since presently developed water resources in the study area are at or approching full utilization, any increased supply may result in an increased cost for water to municipal users. Water conservation may become more attractive because of the increase in costs, but it is also one of the strategies which can be utilized to minimize the capital cost of projects.

The questions posed are:

1. Is a combination of water pricing and conservation incentives an acceptablke method of implementing a water conservation strategy?
2. Would the enforcement of regulations limiting water consumption be an acceptable component of a water conservaion strategy?
3. Do you know of any economically viable method of reducing water consumption on your premises?
4. Since effluent irrigation has been identified as one method of more fully utilizing available water supplies, thus reducing water demand, should effluent irrigation be promoted even though it may be marginally more expensive than continuation of conventional discharges of effluent from lagoons?

The responses may be summerized as follows:

1. The two-thirds positive response to the water pricing issues indicates an acceptance of water as a commodity which is responsive to pricing and recognition that it would be appropriate for municipal govermnments to initiate effective conservation programs with an increase in water prices to finance such activities.
2. The 49% positive response to the regulation question indicates that regulation, which is the most frequently used water conservation tool, is maginally acceptable.
3. Only 135 indicated that they know conservation techniques which could be implemented in their operation. While this is a low positive response it should not be surprising because there are virtually no wet industries in the study area and therefore the industries do not lend themselves to recyling and other accepted conservation techniques.
4. the greater than 69% positive response to the effluent irrigation question was surprising, particularly given the wording of the question which indicated there would be costs associated with the adoption of effluent irrigation. This not only indicates a strong conservation ethic, but probably gives the municipalities a strong message, pariclularly when considered in combination with question one.

VII.3 Water Conservation options Considered

Following is a brief summary of the major conservation options reviewed and recommended by the Task Force to be included in the Water Conservation Strategy:

1. Education - at all levels of society but particularly in the schools
2. Recycling - use of final cell sewage lagoon discharge to irrigate, as an example
3. Retention of Existing Infrastructure - the retention of farm droughts for fire protection is only one example
4. Water Pricing - the setting of water rates to discourage waste is a proven water conservation initiative
5. Maintenance of Municipal Infrastructure - maintaining pipeline networks to keep losses to an acceptable minimum - say 10%

VI.4 Current Initiative

The Water Conservation Program in the Pembina Triangle is ongoing. The recently announced Federal Green Plan includes provisions for pilot projects in municipalities to evaluate the effectivness and the economic impact of water conservation initiatives. Green Plan officials have been working with municipal officals in one of the larger communities in the Pembina Triangle. Plans are yet not complete; however an action plan is being developed which will include in its first phase investigations to locate and seal leaks in the water distribution system in the community and an aggressive public education program to reduce consumption at the household level. Depending on the results, municipal by-laws may be changed to encourage the use of reduced water using household applicances in new residence and retrofitting existing homes. It is expected that the other communities will monitor the sucess of this first Green Plan Project initiative and , depending on its sucess, will implement their own programs. The fact that a community within the Pembia Triangle voluntereered to be a "pilot community water conservation project" is a further demonstration of the committment of the people in the region to conserve water to the extent possible.

LAND QUALITY AS AFFECTED BY THE WASTEWATER TREATMENT LAGOON AND DISCHARGE ROUTE AT ST.CLAUDE, MANITOBA

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INTRODUCTION

Since the construction of a sewage lagoon system in 1970, the Environmental Control Branch had received a number of complaints with respect to offensive odour and seepage. Landowners of adjacent lands claimed that seepage from the lagoon was killing their crop; they could not grow anything in the affected areas. Over the years,a number of technical reports to the Department of Environment provided data on water quality, effluent analysis, soil analysis, and relative elevations of the effluent in the lagoon, ditch system and adjacent land. However, consultants have differed in the interpretation of causes and source of salinization, extent of area affected, and possible remedial measures. As a result, in 1989, the Environmental Control Branch requested the participation of the Canada-Manitoba Soil Survey to conduct an independent study on land surrounding the sewage treatment facility. The objectives were to provide recommendations on environmental mitigation, if possible, or to evaluate a number of possible scenarios in terms of future environmental impact and possible remedial measures (Michalyna,1989). This paper is based on this study and deals with the effect of saline (sodic) wastewater on land quality adjacent to the lagoon and discharge route.

BACKGROUND INFORMATION

Lagoon and Wastewater

The sewage lagoon system serving the Village of St. Claude was constructed in 1970. The system consisted of two aerated cells followed by a large storage cell. The two aerated cells were lined with clay. The storage cell was not lined with impervious material, having been constructed with insitu material. There were two problems associated with the sewage treatment system; excess seepage and saline wastewater. The sewage lagoon allows excessive seepage due to construction with improper material (Manitoba Environment, 1988).
Excess salinity in wastewater was indicated by high levels of salt in the lagoon, 1978 to 1987, which ranged from 4,760 to 12,800 micromhos per cm conductivity; sodium levels of 1100 to 2275 mg/L and chloride 1160 to 2900 mg/L (Manitoba Environment, 1988). High levels of sodium and chloride were due to the contribution from several users who were drawing water from deep wells.

Municipal Water

Salt concentrations in municipal water supply in 1987 and 1980 were 795 and 949 micromhos/cm conductivity, respectively; the average sodium was 29.6 mg/L and chloride was 12.5 mg/L.

Groundwater at a dug well 1/4 mile E of the lagoon had a conductivity of 1720 umhos/cm, sodium 33.5, chloride 31.3, sulfate 207.5 mg/L (Manitoba Environment, 1988).

Stratigraphy

The geological materials at the lagoon site are dominantly sandy deposits and greater than 3 m thick (Manitoba Environment, 1988):

- sand SM, extending to depths of 3.3 m (min.) and 5.1 m (max.) with 72 to 79% fine sand, 21 to 28% silt plus clay content, and a coefficient of permeability from $4.5x10^{-4}$ to $1.97x10^{-5}$ cm/sec.

- clay CI to CH, with clay content 20 to 65% and silt content 32 to 69% .

Relative Elevation of Area

The elevation in the general area surrounding the lagoon (mainly section 14 and 15 of T8 R7W varies from 307 m a.s.l. in the SW corner of section 15 to approximately 299.5 m a.s.l. in the NE corner of section 14. The general slope of the land in proximity to the Railway ROW is 7 m in 3.30 km (0.2%).

The lagoon site occurs in a landscape between 303 and 304 m a.s.l. There is no natural drainageway within the area; soils of the area are sandy and most of the precipitation moves to the groundwater. Groundwater flow is northeasterly, perpendicular to the contours.

Analysis of Soil from Other Areas in the Township

Samples taken for estimates of salinity during the soil survey of the RM of Grey (Michalyna, 1988) were few because salinity within the Township 8-7W was seldom encountered. Sampling sites were mainly depressional or adjacent to small depressions within the landscape. ECe ranged from 1.6 to 7.2 dS/m on the sandy or loamy soils. Soil samples from areas above the 305 m a.s.l. were few; ECe were less than 2.6 dS/m. The dominant cations were calcium and magnesium; sodium adsorption ratio (SAR) were commonly below 1.7. The dominant anion was sulfate; chloride ion was low at less than 2.0 m.e./l.

Based on drill information to approximately 3 m depth in selected areas throughout the township, sandy deposits had an EC less than 1.2 dS/m, in most instances 0.2 to 0.5 dS/m. Where a higher silt or clay content was encountered, the ECe was more variable ranging from 0.4 to 3.1 dS/m.

METHODS

Available information pertaining to the seepage problem at St. Claude Lagoon site was reviewed.

Pertinent data was plotted on prepared field sheets at a scale of 1:5,000; this included soil and water conductivity, sodium level, chloride level and sulfate level (anions and cations were expressed as m.e./l).

Soil conductance measurements (mmhos/m) were taken with a Geonic EM38 conductance meter in the vertical (V) and horizontal (H) position. Seven sites were selected around the lagoon to obtain additional data on salt level, water table and moisture distribution.
Soil samples were obtained at approximately 50 cm intervals to depths of saturation or lower. Salt analysis were determined by the Provincial Soil Testing Laboratory using standard methods. Shallow well piezometers, consisting of 5.0 cm O.D. aluminum tubes with a fibre glass screen at the base were installed in the auger hole. EC of the water was determined with a portable conductivity meter.

Elevations were taken at the top of the pipe and ground level to determine the water table elevation.

Core samples were taken at regular depth intervals adjacent to the well site (5X and 7X) with steel cylinders of 4.8 cm I.D. and 5.0 cm length. The core sample was weighed in the field on a portable balance. Each core was placed in a tared plastic tray and water was added to the tray to allow soil to reach a saturated condition; excess water at the base was removed with an eyedropper, then the core was weighed. The core was dried at 110°C and weighed. Volumetric field moisture, volumetric saturation and bulk density were calculated.

RESULTS and DISCUSSION

Extent of Salt Affected Area

Based on EM38 Conductance meter data from the V position within the NE 1/4 of 15 and NW 1/4 of 14, Twp. 8, Rge. 7W (Fig.1), the soil salinity was highest within 100 m of the lagoon site and ditches as indicated by isolines of greater than 80 mmho/m (Fig.2); higher readings of 130 to 150 were common in areas of "no growth" adjacent to the fence boundary on the west and north with few localized areas above 150. Areas outside the 60 mmho/m isoline have a soil ECe less than 2 dS/m. Most of the area to the south of the Railway ROW had low conductance below 40 mmho/m (soil ECe had less than 0.7 dS/m). The location of previously reported and present sampling sites, soil conductivities and effluent conductivities of the storage cell are provided in Fig.3.

Type of Salinity

Results of cation and anion analysis conducted on the saturated soil solution for sampling sites in Fig.3 have been reviewed and converted to m.e./l. The ions of importance to this study, mainly sodium (Na) and chloride (Cl) are presented in Fig.4,5, respectively.

Sodium Concentrations

Sodium levels adjacent to the lagoon site and drainage network were high with concentration ranging from 14 to 120 m.e./l (Fig.4). Near surface concentration of sodium were considerably higher than subsurface concentrations, which indicated that salts were concentrating above the water table due to capillarity. On the west side of the lagoon, the higher sodium level extended at further distance as compared to the area north and east of the lagoon.

Chloride Concentrations

Chloride levels adjacent to the lagoon site and drainage network (Fig.5) were high with concentration ranging from 22 to 134 m.e./l. Near surface concentration of chloride are considerably higher than subsurface concentration, similar to the sodium trend. On the west side of the lagoon, chloride concentrations were relatively low at the two most westerly sites (site 2X and (14) Fig.5). The chloride ion is one of the least reactive ions in the system; therefore, it can be used as a water flow indicator. The background or natural chloride concentration were typically less than 2 m.e./l (Michalyna,1989), dug well 1/4 mile E of the Lagoon with 0.9 m.e./l (Manitoba Environment,1988), less than 1.0 m.e./l in the NE 1/4-15-8-7W (Site 2, Nielsen, 1988a).

A zone of chloride influence due to seepage (Fig.5) was estimated from the concentration of chloride from various soil analysis in conjunction with the conductance isolines. Within this zone, the chloride level was greater than 5 m.e./l and increases with proximity of the lagoon or drainage network.

Sodium Adsorption Ratio

Sodium adsorption ratios (SAR) are high in areas adjacent to the lagoon (Fig.4) where sodium concentrations are relatively high (areas of no growth); SAR values range from 2.2 to 16.9. As the SAR values increase, sodium ion acts as dispersion agent on soil colloids (clay and organic matter); dispersed soils lack structure, have reduced permeability, and generally have a higher bulk density. These conditions result in a sealing effect to water flow. Soils tend to remain wet and waterlogged in addition to having appreciable salinity. Even if salinity is not high enough to affect plant growth, the lack of adequate aeration in the root zone becomes a problem.

Salt Levels in Storage Cell

Salt levels in the storage cell have been quite variable since the lagoon became operative in 1970 to May, 1988; the EC has varied from 4.7 to 12.8 mmhos/cm (Fig.3), sodium 47.8 to 92.9 m.e./l (Fig.4), and chloride 32.7 to 81.7 m.e./l (Fig.5). From 1988, EC of the effluent was 2.1 to 2.2 mmhos/cm (Dillon, 1988; Michalyna, 1989); sodium was 24 m.e./l. and chloride was 21.4 m.e./l. SAR was 14.1. Even though the EC has decreased appreciably, the SAR remains high at 14.1. This is probably due to water softener use by approximately one-half the households in the town.

Capillarity, Degree of Saturation and Bulk Density

Field moisture content, bulk density and saturated moisture content were determined at two locations, sites 5X and 7X (Fig.3) to the east of the lagoon. At site 5X, field moisture (Fig.6a) was relatively high in relation to the volumetric saturation. The soil was near saturated condition from approximately 40 cm depth to the water table at 86 cm depth. Within the upper 40 cm depth, moisture content remained relatively high,resulting in poor aeration for optimum growth; air-filled porosity (AFP) was 5 and 3 percent at the 20 and 40 cm depth, respectively. Crops require adequate aeration during all stages of growth, generally greater than 10% AFP within the root zone. AFP refers to the volumetric air content in soil and determined as the difference between the water content at saturation and field moisture. Degree of saturation refers to the ratio of the field moisture to saturation, expressed as percent.

Bulk density was 1.69 to 1.78 g/cc within the 60 cm depth, and less than 1.63 below 80 cm. A similar soil at site 7X (Fig.6b) that was not influenced by salinization and sodic conditions had bulk densities of 1.50 to 1.64 in the root zone.

The moisture contents of the Willowcrest soil at site 7X are provided in Fig.6b. The soil was near-saturation at a depth of 50 cm to the water table at 97 cm depth (AFP of 8% at 50 cm. The field moisture content at depths of 15 to 50 cm was similar to site 5X, but the degree of saturation was considerably lower, 68% at 20 cm (AFP of 22%) as compared to the salinized sodic profile of site 5X.

A slight rise in the water table would drastically affect the moisture content and air-filled porosity in the near surface at site 5X as compared to the soil at site 7X. Also, the effect of a rainfall of 10 mm or more would result in saturation and ponding of the soil at site 5X (and similar salinized sodic soils adjacent to the lagoon). The moisture distribution of the soil at site 7X appears similar to a typical Willowcrest soil (Fig. 6c) from the NW quarter of section 22 (approx. 2 km north of the lagoon). Following irrigation or adequate rain to wet the soil, the excess moisture will drain downwards toward the water table. At some height above the water table, the water no longer drains freely but is affected by capillary forces (approx. at 55 cm depth) and moisture content gradually increases to saturation. Near the water table, there is a saturated capillary zone of approximately 30 cm above the water table (at 135 cm depth). The capillary effect was approximately 80 cm.

The presence of appreciable salts in the soils would result in an increase of the maximum capillary height as compared to a non-saline soil. As a result, in a biological and soil system where water loss would occur from the soil surface and root zone, by evaporation and evapotranspiration processes, capillary water would move upward and transport soluble salts into the rooting zone. The deposition and accumulation of salts in the root zone eventually affects the growth of plants.

Soil Properties and Crop Growth

A general map has been prepared to indicate the degree to which soil properties have been affected or altered by seepage (Fig.7), which in turn could be interpreted for crop growth or possible performance. The terms are strongly (S), moderately (M), and non affected (N). A few arbitrary limits are provided as follows:

S - strongly affected soils - characterized by relatively high soil conductance commonly greater than 90 mmho/m, relatively high moisture content and capillarity to near surface, relatively high SAR value 2.2 to 14.6, chloride generally greater than 40 m.e./l, subject to ponding and poor aeration. Growth of seeded crop or common grass-legume mixture is minimal to absent over most of the area.

M - moderately affected soils - characterized by relative soil conductance commonly between 50 to 90 mmhos/m, ECe commonly greater than 2 dS/m in the root zone, chloride content in soil extract commonly greater than 5 m.e./l and SAR value 1 to 2. Growth of seeded crop or common grass-legume mixture is attenuated toward the limits of negligible crop growth and will vary from apparent normal growth to poor growth. More sensitive crops such as corn would exhibit an attenuation over a considerable portion of this zone, while crops with moderate salt tolerance (or tolerate periods of dcreased aeration) would show a narrower zone of attenuation of crop growth adjacent to minimal or no growth area.

N - non affected soils - characterized by low chloride below 2 m.e./l, a dominance of sulfate ion, soil conductance commonly less than 60 mmho/m, SAR less than 1. Crop growth is normal for the area.

The estimated area (ha) of the strongly affected (S) and moderately affected (M) soils are as follows: NE1/4 15 had 4.0 of S, 7.8 of M; NW1/4 14 had 5.6 of S, 6.3 of M.

The strongly affected soil areas would correlate with the general condition of "no growth area" (Nielsen, 1988a,b) or "bare area" (Manitoba Environment,1988); moderately affected soil areas would correlate to some extent with the term "restricted growth" areas (Nielsen,1988). The term "attenuated growth area" is a preferable crop growth descriptor within this moderately affected soils area.

SUMMARY and CONCLUSIONS

1. Seepage will continue to the surrounding area with effluent that has an electrical conductivity of approximately 2.1 dS/m and an SAR value of 14.1 .

2. Operation of the storage cell with effluent at or above 303.5 m a.s.l. will provide sufficient head for seepage to the drainage network and adjacent lands.

3. Water table in lands adjacent to the ditches and drainage network would be sufficiently shallow to influence capillarity to the surface; salinization and sodic effect will continue to spread gradually beyond the present area of strongly affected soils (no crop growth area).

4. Present areas of "no crop growth" have been affected by the increase in sodium level; the soils have appreciable salts (ECe in the root zone of 5 to 14 dS/m and SAR of 2.2 to 14.1 values). The bulk density has increased considerably from less than 1.60 to values of 1.75 -1.78 g/cc. Capillarity effect extends to the surface resulting in decreased air porosity. Ponding and near saturation conditions after rain showers is common leading to inadequate air-filled porosity and reducing conditions (oxygen deprivation for crops). Permeability would be considerably less in the near-surface soil layers where the bulk densities are higher as compared to similar non-affected soils.

5. Near-surface groundwater salinity would decrease in the immediate vicinity of the lagoon, but the continued hydraulic head would result in expansion of the present deeper, more saline groundwater (EC 4.1 to 7.4 dS/m) from the lagoon area, the ditch and drainage network.

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DESIGN AND STARTUP OF THE PENTICTON WASTEWATER TREATMENT PLANT

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City of Penticton

INTRODUCTION

The Penticton Advanced Wastewater Treatment Plant discharges into the environmentally sensitive Okanagan Valley lake system and is designed for biological nitrogen and phosphorus removal. In March, 1987 the City of Penticton initiated a program for the upgrading of its conventional activated sludge plant (with chemical P removal) to an advanced wastewater treatment plant incorporating biological nutrient removal. Phosphorus had been identified as the principal growth limiting nutrient in the receiving waters and the project forms part of a regional strategy to limit the discharge of phosphorus to the lakes. Complete nitrification was also required due to a concern about ammonia toxicity of the effluent to fish. The upgraded plant was commissioned in November, 1990, and serves a summer design population of 53,000. The corresponding design average dry weather flow (ADWF) is 18,200 m^3/day (4.0 IMGD). Effluent from the plant is either discharged through an outfall into Okanagan Channel which joins Okanagan Lake to Skaha Lake, or used to irrigate the municipal parkland and the golf course.

PROCESS SELECTION AND DESIGN

The influent organic and nutrient loading and associated wastewater characteristics used in the design of the plant are as follows:

Parameter	Summer	Winter
BOD_5 (kg/day)	3,640	2,660
(mg/L)	200	200
Total Nitrogen (kg/day)	820	600
(mg/L)	45	45
Total Phosphorus (kg/day)	93	68
(mg/L)	5.1	5.1

The plant was designed to meet the following effluent quality objectives:

BOD_5	20 mg/L
Suspended Solids	5 mg/L
Ammonia Nitrogen	6 mg/L (as N)
Total Phosphorus	0.5 mg/L (as P)
Fecal Coliforms	< 2.2/100 ml

The plant was designed to achieve phosphorus removal through enhanced biological phosphorus removal. A backup chemical feed system was also provided for use in the event that the plant cannot achieve its effluent phosphorus limit (1.0 mg/L as P) biologically. The above effluent quality objectives implied that nitrification must also be achieved at all times. Although there was no effluent total nitrogen requirement in the discharge permit, it was decided to also incorporate denitrification into the process design for the following reasons:

1. Nitrate entering the anaerobic zone of the process has a detrimental effect on the enhanced biological phosphorus removal mechanism. By optimally reducing effluent nitrate concentration through denitrification, the possibility of nitrate entering the anaerobic zone of the process is minimized.

2. Approximately 63% of the oxygen required for nitrification be recovered through denitrification. This results in significant reduction (15-20%) of the plant oxygen requirement.

3. Approximately 50% of the alkalinity lost in the nitrification reaction can be recovered through denitrification. This results in a more stable plant pH.

4. The presence of a denitrification zone improves the sludge settling characteristics.

Enhanced biological phosphorus removal is a mechanism in which organisms in the activated sludge process take up a greater mass of phosphorus from the wastewater than that which they require for basic metabolic purposes. While the sludge mass from a conventional activated sludge process has a dry weight percent P content of approximately 1.5%, sludge from a plant designed for enhanced biological phosphorus removal has a dry weight P content of 3-5%. Research carried out worldwide in the last decade has clearly indicated that in order for the enhanced biological phosphorus removal mechanism to reliably occur in an activated sludge process, the following two conditions are required:

1. At some point in the process, the organisms must be subjected to true anaerobic conditions i.e. the absence of both dissolved oxygen and nitrate.

2. A certain minimum mass of simple carbonaceous substrate, primarily short chain volatile fatty acids, or VFA's, such as acetic and propionic acids must be made available to the activated sludge mass in the anaerobic zone.

A detailed biochemical description of the enhanced biological phosphorus mechanism is presented by Comeau et al. (1986) and Wentzel et al. (1990). Briefly, the creation of a zone in the bioreactor in which the above two conditions are met gives organisms involved in the enhanced biological phosphorus removal mechanism a competitive advantage over normal activated sludge organisms. In the anaerobic zone of the process, these organisms store the VFA's intracellularly in the form of poly-B-hydroxybutyrate, or PHB. This storage mechanism is mediated by the breakdown of the stored polyphosphate, resulting in P release which serves to re-establish the proton motor force (PMF) of the cell, and energizes the substrate for storage. Under subsequent aerobic conditions, the PHB is consumed by the organisms to produce energy for the storage of inorganic P in the form of polyphosphate reserves. It is from this point in the process that the phosphorus is removed in the waste activated sludge. By increasing the availability of VFA's in the anaerobic zone of the process, the carbon storage is increased and the P removal characteristics of the process are enhanced.

Short chain VFA's are the principal intermediate product in anaerobic digestion, which is a 2-stage reaction. By subjecting the primary sludge to the acid fermentation stage, VFA's require for enhanced biological phosphorus removal can be produced onsite at a wastewater treatment plant. The fermenter design used at Penticton was based on the University of British Columbia (UBC) Primary Sludge Fermentation System (Figure 1) described by Rabinowitz et al. (1987). Primary sludge is pumped from the primary clarifiers to a completely mixed acid fermenter. The fermenter mixed liquor flows by gravity back to the influent end of the primary clarifiers. The VFA-rich primary effluent flows by gravity to the activated sludge bioreactor. Waste primary sludge from the fermenter is pumped to the anaerobic digesters. The fermenter hydraulic retention time (HRT) is a function of the fermenter volume and the primary clarifier underflow pumping rate. The fermenter solids retention time (SRT) is a function of the sludge wastage rate. By using these two control parameters, the fermenter VFA production can be optimized for a wide variety of operating conditions.

The process configuration selected for the design of the activated sludge bioreactor at Penticton was the Modified UCT Process (Figure 2). This configuration is considered to be the most flexible for creating the prerequisite anaerobic conditions for the enhanced biological phosphorus removal mechanism to operate. Furthermore, it is capable of achieving system nitrogen removals

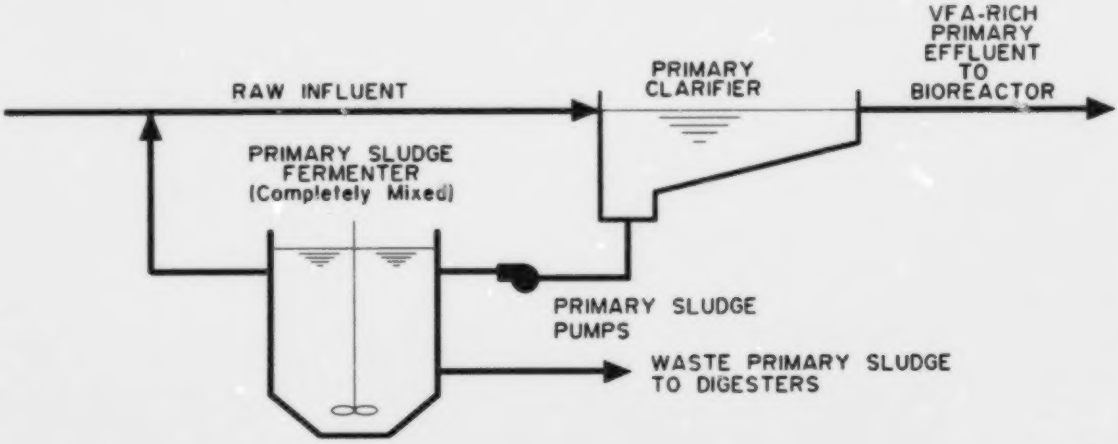
RAW INFLUENT
PRIMARY CLARIFIER
VFA-RICH PRIMARY EFFLUENT TO BIOREACTOR
PRIMARY SLUDGE FERMENTER (Completely Mixed)
PRIMARY SLUDGE PUMPS
WASTE PRIMARY SLUDGE TO DIGESTERS

FIGURE 1 U.B.C. PRIMARY SLUDGE FERMENTATION SYSTEM

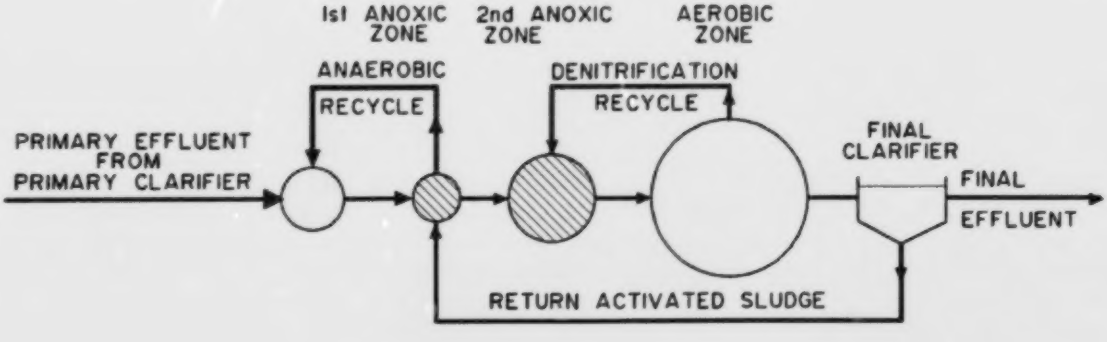

FIGURE 2 MODIFIED UCT PROCESS CONFIGURATION

greater than 80% through nitrification and denitrification. The Modified UCT process configuration consists of the following four reaction zones in series:

1. The anaerobic zone receives the VFA enriched settled sewage from the primary clarifiers and the nitrate-free anaerobic recycle from the 1st stage anoxic zone. It is in this zone that the required environment for enhanced biological phosphorus removal (absence of DO and/or nitrate and the presence of VFA's) is created. In general, there should be no detectable nitrate in the zone and the supernatant phosphorus concentration at the end of the zone will be significantly greater than that in the influent wastewater as a result of anaerobic phosphorus release.

2. The 1st stage anoxic zone receives and denitrifies the return activated sludge (RAS) recycle flow from the secondary clarifier, thus ensuring that no nitrate enters the anaerobic zone.

3. The 2nd stage anoxic zone receives the denitrification recycle from the end of the aerobic zone. Its principal function is to denitrify the majority of the nitrate produced in the aerobic zone of the process. In addition, a significant degree of phosphorus uptake and carbon removal occurs in the zone.

4. The aerobic zone of the process is the largest reaction zone, making up the balance of the effective bioreactor volume. The aerobic zone is made up of cells receiving both mixing and aeration. In this zone carbon and phosphorus removal are completed and all of the process nitrification occurs. Phosphorus is removed from the process in the waste activated sludge.

UNIT PROCESSES

The upgraded plant consists of the following unit processes:

- A headworks building containing three Archimedes screw pumps, a mechanically raked bar screen, and a "Pista" type swirl degriter and grit washer.

- Six rectangular primary clarifiers equipped with chain and flights, manually operated scum troughs and a scum decant tank.

- Two flow equalization basins whose principal function is to ensure a steady organic loading and hydraulic flowrate to the bioreactor during the nightly low-flow period.

- Two completely mixed primary sludge fermenters whose principal function is to generate the short chain VFA's required for enhanced biological phosphorus removal.

- Two eleven-cell bioreactor modules designed for optimal biological phosphorus and nitrogen removal. Although nitrogen removal was not required, it was included for a number process considerations (improved sludge settlability, alkalinity recovery, reduced oxygen requirements, etc.) The bioreactor was designed around the Modified UCT process configuration and has the following four reaction zones in series: anaerobic, 1st stage anoxic, 2nd stage anoxic and aerobic. Mechanical mixing is provided in all anoxic cells and both mixing and aeration is provided in all aerobic cells. Internal recycling between the reaction zones is provided by low-lift axial flow propeller pumps.
- A blower building containing three centrifugal blowers to provide air to the aerobic zone of the bioreactor.
- Two 27.8 m diameter secondary clarifiers equipped with centre drive scraper mechanisms, and scum skimmers.
- An effluent filtration building housing 3 single media filters equipped with automatic traveling bridge backwash systems.
- Chlorination/dechlorination tanks.
- Two anaerobic digesters for digestion of waste primary sludge from the fermenters.
- A sludge thickening and dewatering facility having a combination gravity belt thickener/belt filter press with associated polymer feed equipment for thickening and dewatering of the digested and waste activated sludges.
- Two lined effluent storage basins capable of storing either the primary effluent, or the treated effluent before or after filtration, chlorination and dechlorination. These basins also serve as the reservoir for the effluent spray irrigation system.
- A treated effluent pumpstation with three vertical turbine pumps. The treated effluent is used to irrigate local parkland and a golf course.
- A chemical storage and feed facility capable of feeding sodium acetate or alum solutions to the bioreactor in the event of process excursions with respect to biological phosphorus removal, and sodium carbonate or slaked lime in the event of a nitrification failure due to a low pH condition in the bioreactor.

The plant was designed to have identical dual process trains, or modules, by splitting the main process stream from upstream of the primary clarifiers to downstream of the secondary clarifiers. This flow splitting can facilitate optimization of the plant by affecting operational changes to the primary clarifiers, fermenters or bioreactors of one module and comparing the resultant performance with the other module.

PLANT STARTUP AND COMMISSIONING

The upgraded plant was started up in June, 1990, and made fully operational in November, 1990. During start-up, the bioreactor was seeded using sludge from the existing activated sludge process, so nitrification and denitrification was established within a short time. The fermenter was not seeded during start-up, but VFA production began spontaneously within a few weeks, and the pickle liquor addition (for chemical P removal) was discontinued as soon as biological phosphorus removal began to take place.

INITIAL OPERATING EXPERIENCE

Graphs of the raw influent and final effluent COD, TSS, nitrogen, phosphorus and coliform concentrations for the period January - May, 1991 are presented. Since plant start-up, the plant has achieved highly efficient levels of COD, TSS and nitrogen removal, with the effluent quality consistently being significantly better than that of the plant requirements. The final effluent COD concentration is consistently less than 30 mg/L (avg. = 18.5 mg/L), and the BOD_5 concentration is consistently less that 10 mg/L. The suspended solids concentration in the secondary clarifier effluent is generally less than 10 mg/L, and the final effluent suspended solids concentration after filtration is less than 4 mg/L (avg. = 2.5 mg/L). The sludge settling characteristics are good with SVI values generally below 150.

The plant achieves virtually complete nitrification all of the time, with the final effluent ammonia nitrogen concentration being generally less that 1.0 mg/L (avg. = 0.4 mg/L). Occasionally, higher effluent ammonia nitrogen concentrations of up to 2.0 mg/L have been observed, but these have been associated with improper dissolved oxygen control in the bioreactor. The effluent nitrate nitrogen concentration ranges between 4 and 6 mg/L (avg. = 5.0 mg/L), indicating that the plant achieves a system nitrogen removal that is > 80%. Although the system phosphorus removal characteristics have met the effluent quality objectives (average ortho-P concentration = 0.7 mg/L), some optimization of the plant is still required in order to consistently achieve a high level of phosphorus removal. The effluent phosphorus concentration has exceeded 1.0 mg/L at times and this has been attributed to one or both of the following causes:

1. Insufficient VFA production due to poor solids level control in the primary sludge fermenters. It appears as if approximately 15-20 mg/L of VFA (measured as acetic acid per litre of influent) are required in the anaerobic zone of the process to ensure reliable enhanced biological phosphorus removal occurs.

2. Excessively long bioreactor sludge age due to operational problems with the gravity belt thickener. Operational problems being encountered are insufficient hydraulic loading capacity (approximately 65% of that specified), and a solids capture rate of approximately 70%. A long bioreactor sludge age may cause cell lysing and subsequent release of the stored polyphosphate in the aerobic zone and secondary clarifiers, causing a deterioration in the system phosphorus removal.

CONCLUSIONS

Initial operating experience at the Penticton Advanced Wastewater Treatment Plant (the plant had been fully operational for approximately 7 months at the time of writing) has led to the following conclusions:

1. In a plant designed for biological nutrient removal (BNR), efficient enhanced biological phosphorus removal is dependent on a sufficient quantity of short chain volatile fatty acids being supplied to the anaerobic zone of the bioreactor. It is estimated that approximately 15-20 mg/L of VFA is required for this purpose.
2. Efficient biological phosphorus removal is dependent on good bioreactor SRT control, with excessively long SRT's resulting in P release due to cell lysis in the aerobic zone and secondary clarifiers.
3. Virtually complete nitrification, and system nitrogen removals of over 80% are consistently achieved at the plant.
4. Final effluent BOD_5 and suspended solids concentrations of below 5 mg/L are consistently achieved at the plant.
5. The sludge settling characteristics are good in a plant designed for biological nutrient removal, with SVI values being generally below 150.

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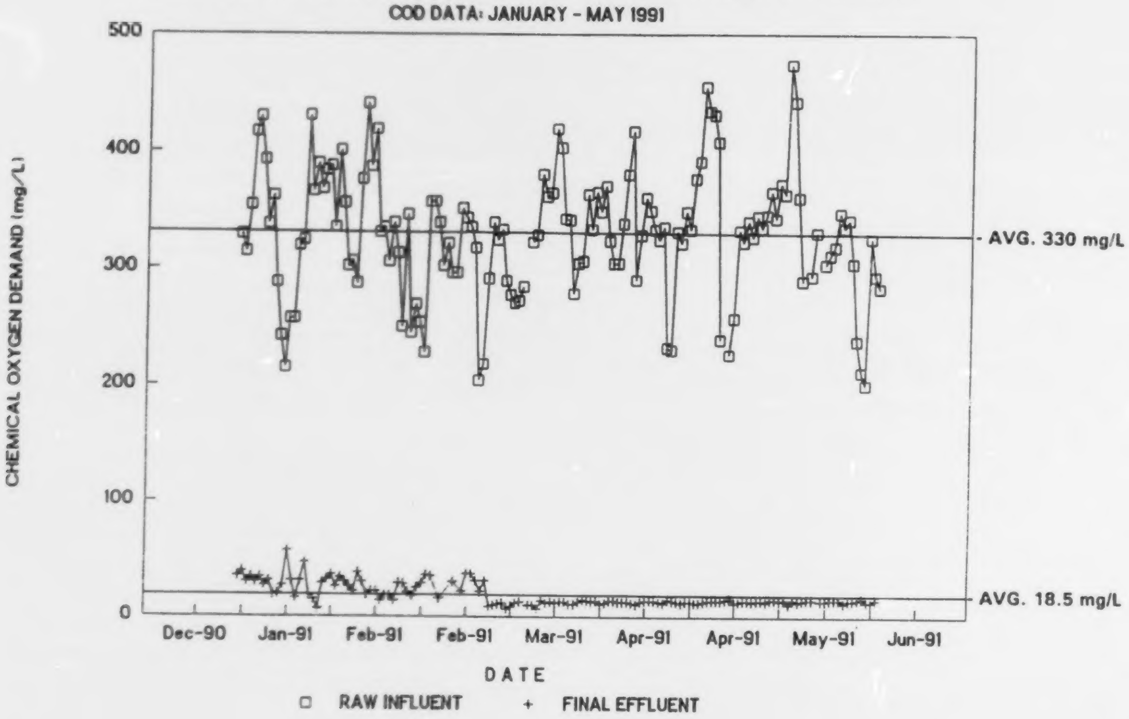
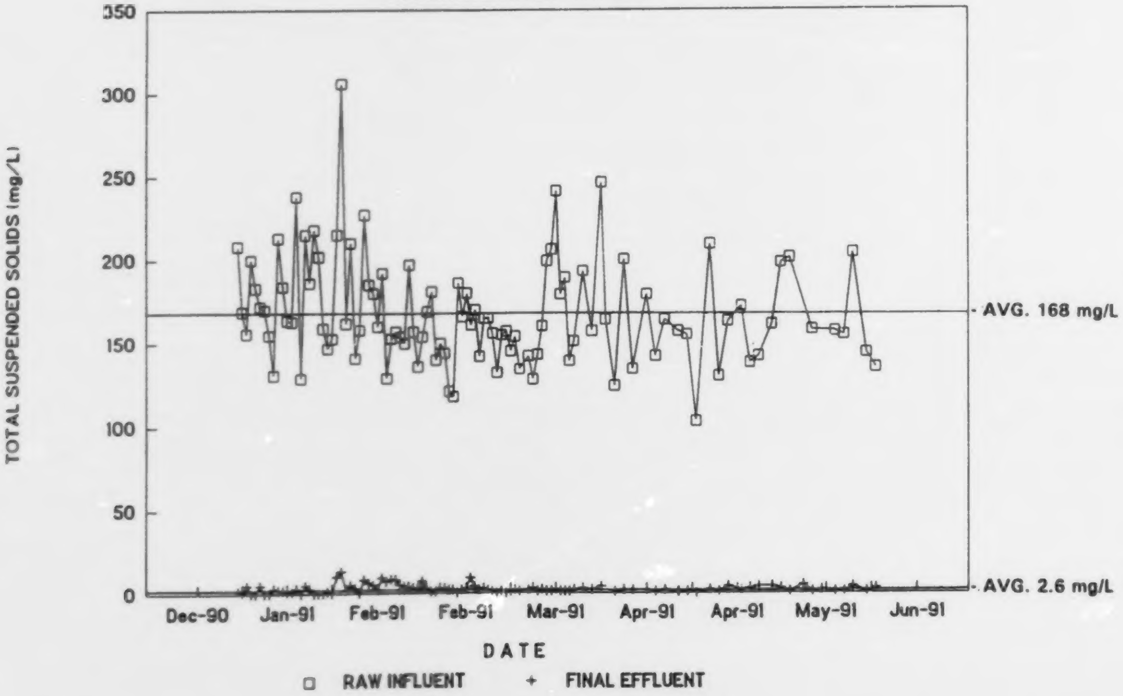
PENTICTON WASTEWATER TREATMENT PLANT

COD DATA: JANUARY - MAY 1991



PENTICTON WASTEWATER TREATMENT PLANT

TSS DATA:JANUARY-MAY 1991



PENTICTON WASTEWATER TREATMENT PLANT

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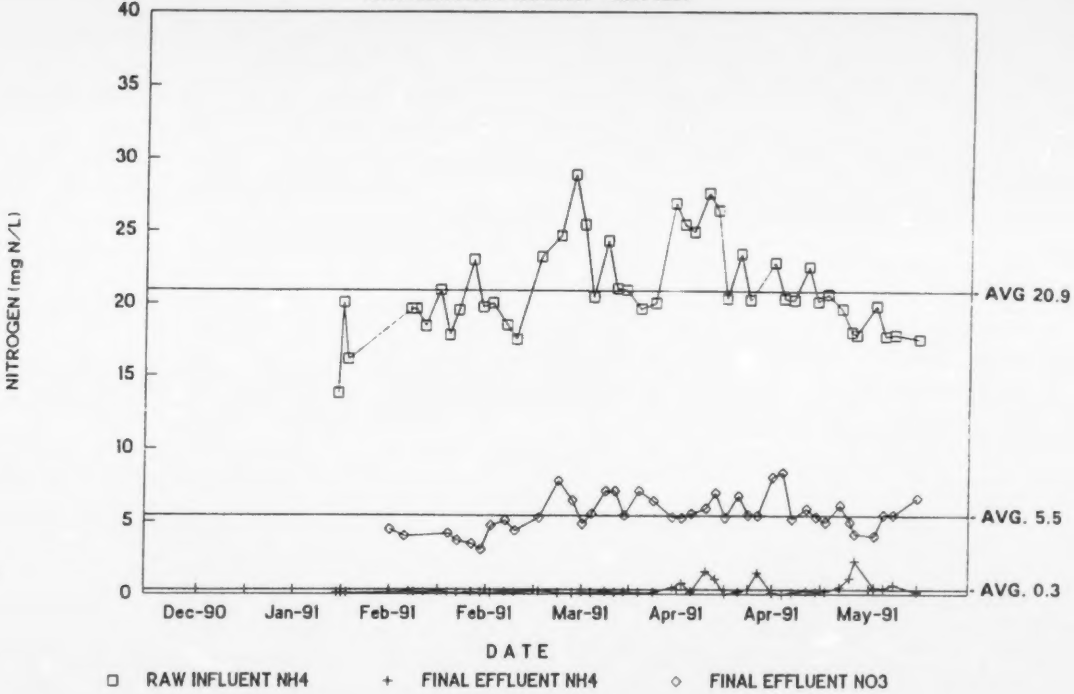

PENTICTON WASTEWATER TREATMENT PLANT

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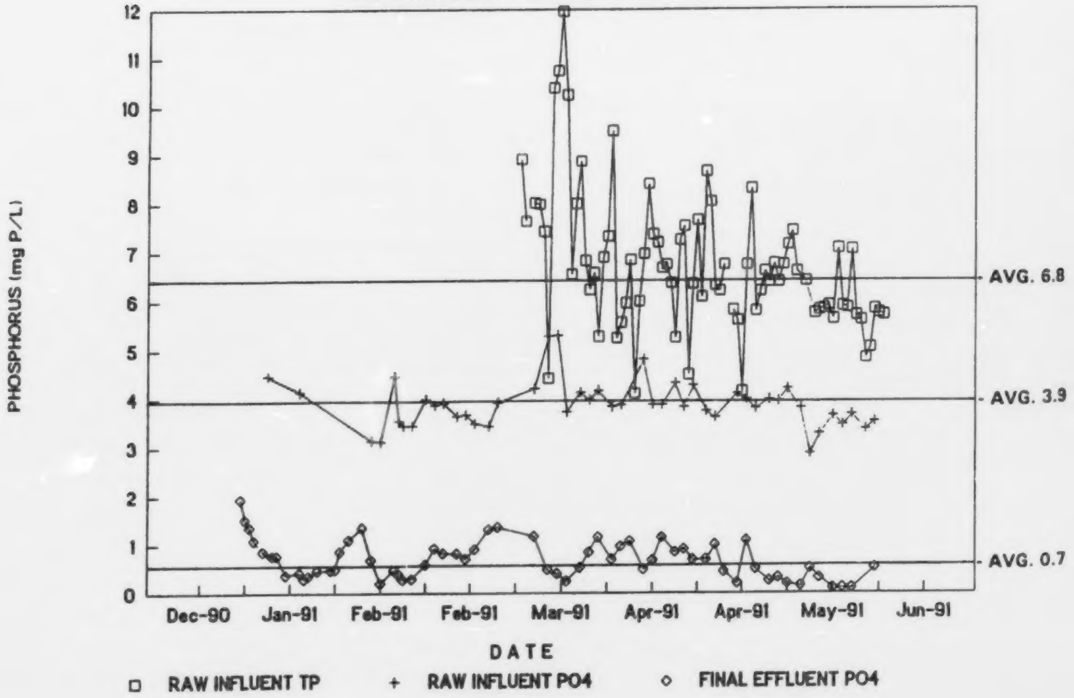

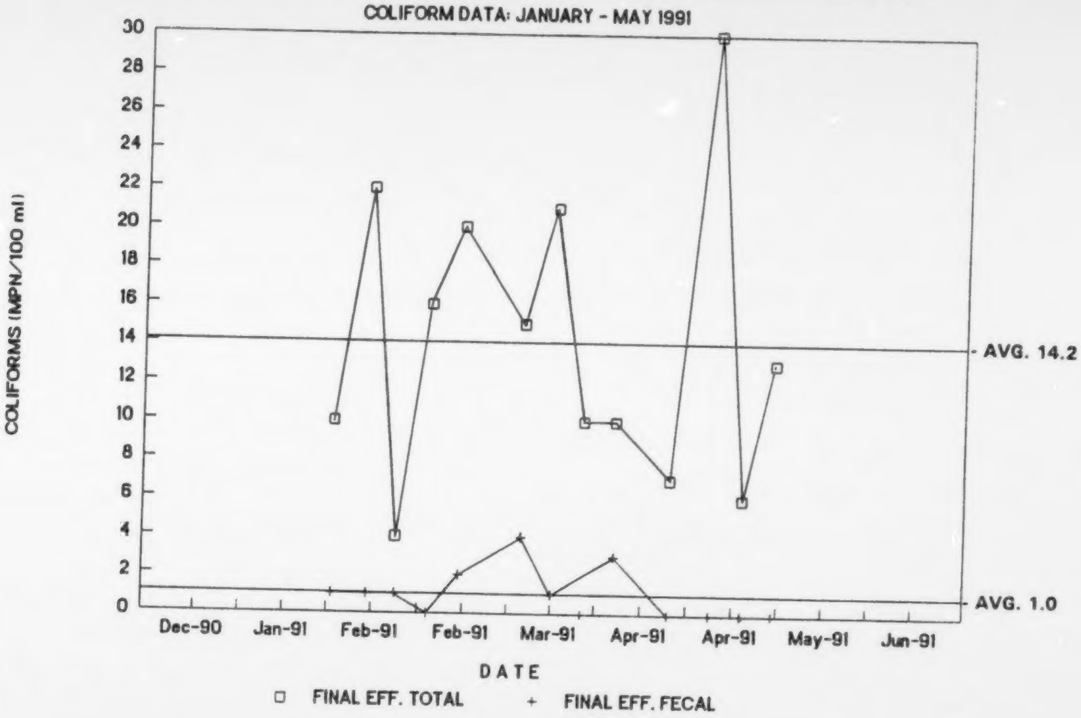

PENTICTON WASTEWATER TREATMENT PLANT

COLIFORM DATA: JANUARY - MAY 1991

DESIGN AND START-UP OF THE MEDICINE HAT WASTEWATER TREATMENT PLANT

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Stanley Associates Engineering Ltd.

INTRODUCTION

In 1978, the City of Medicine Hat initiated a four phase program to replace and/or upgrade their existing aerated lagoon sewage treatment facility.

The following program was implemented:

Phase I:	Study of treatment requirements, design and construct a mechanical primary treatment plant. The effluent from this plant to discharge to the existing aerated lagoons.
Phase II:	Monitor the performance of the new primary plant and lagoons. Evaluate the results and the need for further secondary treatment.
Phase III:	Subject to the evaluation carried out in Phase II, design and construct new and/or upgraded secondary treatment facilities
Phase IV:	Removal of nutrients if this ultimately becomes a requirement.

In March of 1981 the primary plant was completed and placed into operation. The operation of the system was monitored until mid-1984 when the study for new secondary facilities started. At this time it was apparent that phosphorus removal would become a requirement in the near future. As a consequence, the removal of phosphorus was included in the Phase III component of the program. By the spring of 1990, new secondary treatment facilities, including phosphorus removal and disinfection, were placed into operation.

PLANT DESIGN CRITERIA

In studies carried out for the design of the primary plant, a twenty year design period was selected. The population, including the Town of Redcliff, was projected to grow from about 44,000 to 71,000 during this time frame, .

On the basis of these population projections, the design average daily flow for the plant was estimated to be 36,565 m^3/d.

During the subsequent study carried out for the secondary facilities, these projections were reaffirmed except that the time frame for reaching the design population level was extended to the 2001 to 2005 period.

The design was based on the following influent characteristics:

Flow Volumes:	Average	- 36,565 m^3/d
	Peak	- 73,130 m^3/d

Influent Characteristics:	Biochemical Oxygen Demand (BOD) - 170 mg/L
	Suspended Solids (SS) - 150 mg/L

Plant records during the monitoring period indicated the following characteristics for design of the secondary plant.

Primary effluent	BOD - 132 mg/L
	SS - 80 mg/L

During the secondary study the effluent requirements were established as follows:

BOD - 20 mg/L
SS - 20 mg/L
DO - 2 mg/L
Include provision for phosphorus removal and disinfection. (These were subsequently incorporated as a requirement.)

PRIMARY PLANT

The primary treatment plant includes the following units:

Headworks: Contains two sets of bar screens and two aerated grit removal chambers.

Primary Clarifiers: Three primary clarifiers, each 21 metres in diameter, have been provided including provision for one future clarifier.

Sludge Handling: Primary sludge is thickened to 6% to 7% in an 8 metre diameter gravity thickener and then dewatered on a 1.5 metre wide belt filter press. The dewatered sludge, averaging 26% moisture content (MC), is then hauled to the nearby sanitary landfill site.

Effluent Pumphouse and Aerated Lagoons: The primary effluent is pumped via a submersible pump lift station to the aerated lagoon system. When the primary plant went into operation the lagoon consisted of four aerated cells each providing about 1.9 day retention at design flow followed by a final settling cell having 4 days retention.

Administration Building: An administration building was included with this phase of construction and provided a superintendent's office, laboratory, lockers, showers, garage and workshop.

The layout of the primary plant including the aerated lagoon system is shown on Figure 1.

PLANT PERFORMANCE DURING THE MONITORING PERIOD

Results obtained during the plant performance monitoring period showed that the existing aerated lagoon system would have to be either upgraded or replaced with new secondary treatment facilities. Lagoon effluent BOD and SS during the period were as follows:

BOD - ranged from 20 to 57 mg/L
SS - ranged from 21 to 28 mg/L

As a consequence, a study of various alternatives to satisfy the City's secondary treatment requirements began in mid 1984.

SECONDARY TREATMENT STUDY

A number of biological processes, and combinations of processes, were examined in the study. The processes considered were:

- aerated lagoons
- roughing trickling filter and aerated lagoon combination
- trickling filter solids contact process
- rotating biological contactors and aerated lagoon combination

- activated sludge
- rotating biological contactors
- land application

Our analysis of these processes showed that the best alternative for Medicine Hat, on the basis of life cycle costs and performance, would be provided by the trickling filter solids contact process. Economy, ease of operation and a high quality effluent were major attributes noted for the process.

Although the process was relatively new, having been discovered in 1980 as an operating modification of the coupled trickling filter activated sludge system at Corvallis, Oregon, a number of plants had been constructed and some were in various stages of planning, design and construction at that time. A site visit to Corvallis and discussions with the designers confirmed its applicability for Medicine Hat. Thus, the process was recommended and accepted by the City by mid-1986.

THE TRICKLING FILTER SOLIDS CONTACT PROCESS (TF/SC)

In the 1940's, trickling filters were used extensively. By the 1950's, the use of activated sludge became dominant. However, since the development of new plastic crossflow filter media and, more significantly, of the discovery of the TF/SC process in Corvallis, trickling filters have once again become a major process consideration for upgrading existing and constructing new facilities.

The process contains both biological and physical treatment units. It includes a trickling filter, an aerobic solids contact channel, a flocculation cell and secondary clarification.

The TF/SC process is defined by Parker (1987) as having the following characteristics:

1. Soluble BOD (SBOD) is removed primarily by the trickling filter (TF).
2. The solids contact (SC) tank's primary function is to capture and flocculate solids and reduce particulate BOD.
3. Hydraulic retention time (HRT) in the SC tank is less than one hour.
4. Solids retention time (SRT) of the SC tank is less than 2 days.
5. Return secondary sludge (RSS) is mixed with trickling filter effluent rather than primary effluent.

The first step of the process consists of the trickling filter which reduces SBOD and develops solids that can be flocculated. In the solids contact tank, initial flocculation between the finely divided solids in the TF effluent and the RSS is provided. Some additional SBOD reduction may also occur in the SC channels. Flocculation continues in the centre well of the secondary clarifiers where large settleable flocs are formed. The solids are then separated in the secondary clarifiers.

Three process modes of operation are available, as shown in Figure 2. The mode selected depends on the particulate and SBOD removal requirements. Mode 1 is selected if significant SBOD and particulate removal is required from the TF effluent. For primarily particulate removal, Mode 2 is selected. Mode 3, which combines the two aerated tanks - return sludge aeration and aerated solids contact, is the best solution for treating a limited amount of SBOD and particulate matter.

The Mode 3 method of operation was selected for Medicine Hat.

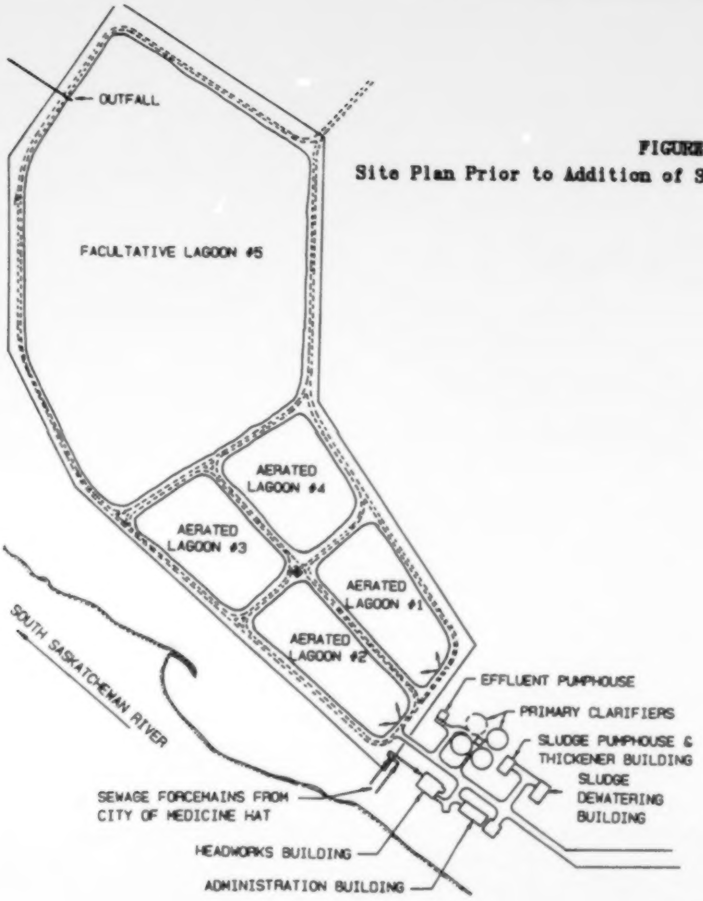

FIGURE 1
Site Plan Prior to Addition of Secondary Treatment Facil

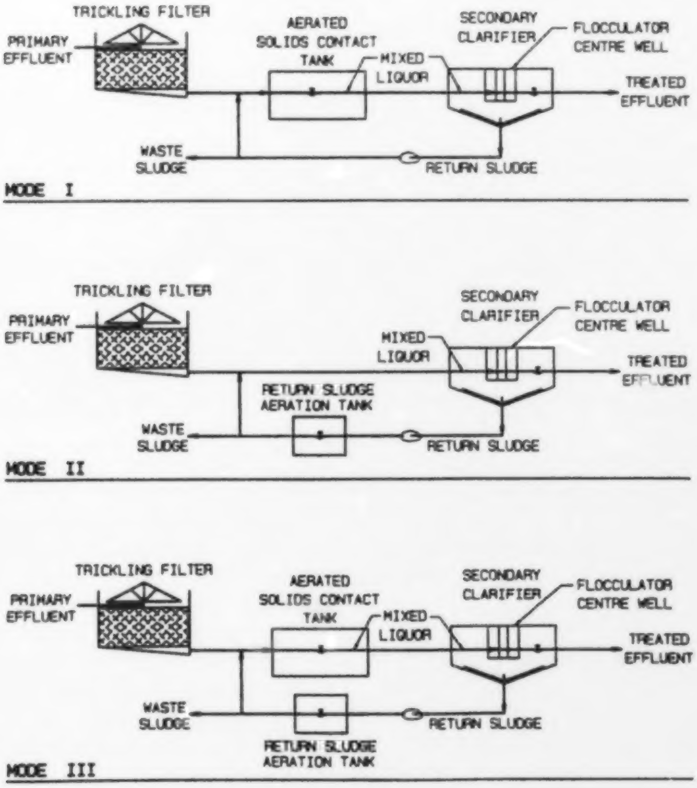

FIGURE 2
TF/SC Process Operational Modes

TF/SC DESIGN PARAMETERS

The concepts and models used for sizing the treatment units for Medicine Hat are described in this section.

Trickling Filters

The filter media volume requirements were estimated using the Schulze equation:

$$\ln \frac{Le}{Lo} = \frac{-k\,\Theta\,D}{Q^n}$$

where

Le = effluent BOD - mg/L
Lo = influent BOD - mg/L
k = removal rate coefficient
Θ = temperature correction factor
= $1.035^{(T-20)}$ where T is the wastewater temperature in degrees C
n = media coefficient
Q = hydraulic loading = USGM/ft^2
D = media depth in feet

1. The effluent BOD was set at 35 mg/L. Le, as used in the Schulze equation, refers to the effluent from the secondary clarifiers for the conventional trickling filter system. However, due to the greater effectiveness of TF/SC process, a value in the range of 30 to 35 mg/L can be used for this parameter. As noted earlier, Lo for Medicine Hat is 132 mg/L.

2. The BF Goodrich Design Manual recommends removal rate coefficient values at 20°C ranging from 0.06 for primarily municipal wastewater with some light industrial waste to 0.08 for purely domestic waste.

   A value of 0.06 was selected.

3. A review of records indicated that the minimum primary effluent temperature was 12°C. Calculations for heat loss through the trickling filters indicated a minimum temperature in the filter of 10°C during sustained ambient cold temperature conditions. Thus, a wastewater temperature of 10° was used.

4. B. F. Goodrich recommend a media coefficient range of 0.5 to 1.0 depending on surface area and transfer efficiency. As these factors increase, the coefficient increases. A value of 0.65 was selected.

5. The recommended minimum hydraulic wetting rate is 0.19 L/s·m^2 (0.7 USGM/ft^2). At this rate solids should be washed continuously thereby preventing large sloughs of solids. Filter flies should also be controlled at this minimum flow rate.

6. The media depth as recommended by B. F. Goodrich should be in the range of 4.9 m to 9.1 m (16 to 30 feet.)

Various combinations of depth and flow rate were applied to determine the optimum media volume.

Predictions for SBOD reduction in the filters were made on the basis of the modified Velz equation. This equation is used routinely by plastic media suppliers and can be stated as follows:

$$\ln \frac{Se}{Sb} = \frac{k\,\Theta\,As\,D}{Q^n}$$

where

Se = SBOD in the filter effluent - mg/L
Sb = influent SBOD = mg/L
k_{20} = first order reaction rate
θ = temperature correction factor
= $1.035^{(T-20)}$
As = media specific surface
D = media depth in feet
Q = feed flux or hydraulic loading
- USGM/ft^2
n = *flow coefficient*

1. The influent SBOD is the blended primary effluent and recycle. Since, at peak flow, there will be no recycle, the primary effluent BOD of 63 mg/L was used.

2. The value of the reaction rate was estimated to be 2.28×10^{-3}. This was based on data obtained at Medford, Oregon, and pilot studies in Garland, Texas. (Parker and Merrill, 1984).

3. The media specific surface area is 98 m^2/m^3 (30 ft^2/ft^3). Various configurations of plastic media are available including vertical, 60° crossflow, 45° crossflow and random. Studies (CH_2M Hill, 1984) have shown that at low organic loading 60° crossflow media will have better BOD removal than other media. Effluent suspended solids will also be less.

   The pilot studies at Garland, Texas indicated that 60° crossflow media will accept 67% more loading than vertical media and obtain equivalent SBOD loading.

4. As indicated in the foregoing, a wastewater temperature of 10°C was used.

5. A flow coefficient of 0.65 was selected.

Application of the above equations resulted in two filters each 23.0 metres in diameter by 7.8 metres deep. The BOD loading at design flow conditions will be 0.74 kg/d·m^3 (45.9 lbs/d/1000 ft^3). In a survey carried out in 1988 (Harrison and Timpany, 1988) average organic loading with plastic filter media ranged from 0.64 kg/d·m^2 to 0.97 kg/d·m^2 (40 to 60 lb/day/1000 ft^3). Soluble BOD loading will be 0.35 kg/d/m (22 lbs/d/1000 ft^3). Effluent SBOD was estimated to be 3.8 mg/L.

Solids Contact Channels

Assuming a MLSS concentration of 2000 mg/L in the SC channels, the SRT provided in the design was 1.5 days. The retention time available in the channels is 45 minutes at design average daily flow including 40% sludge recycle.

Soluble BOD removal in the channels was estimated on the basis of the following model (Matasci et al, 1985)

$$\ln \frac{C}{C_o} = [-k\,\theta\,X]t$$

C = soluble BOD after time t
C_o = soluble BOD of mixed liquor at the beginning of the channel
k = reaction rate coefficient
θ = temperature correction factor
= $1.035^{(T-20)}$
X = MLVSS
t = channel retention time

1. Soluble BOD - C_o was assumed to be 14 mg/L.

2. The reaction rate coefficient used was 2.5×10^{-5} at 20°C

3. The MLVSS used for the design was 2000 mg/L.

The resulting SBOD after 45 minutes was calculated to be 2.8 mg/L.

Two channels were provided - each 44 m long X 5.5 m wide X 4 m deep.

Secondary Clarifiers

Recent design concepts were incorporated in the design of the secondary clarifiers. These included:

- inboard effluent troughs to avoid the vertical density currents occurring at the clarifier wall. This also increases weir length, thereby reducing weir overflow rates.

- flocculating centre well with mixers to promote continued agglomeration of the floc

- deep side wall depth in order to provide good separation of the sludge blanket from the overflow weirs. This also reserves storage capacity for peak flow periods.

For the TF/SC process higher overflow rates can be used than for activated sludge process due to the more settleable floc achieved in the SC channels and flocculating centre wells. The overflow rates selected for Medicine Hat were 23.5 $m^3/m^2 \cdot d$ (580 USGD/ft^2) at average flow to 46.9 $m^3/m^2 \cdot d$ (1160 USGD/ft^2) at peak flow conditions. In the survey carried out by Harrison and Timpany (1988) they found that peak wet weather flow rates of 48.5 $m^3/m^2 \cdot d$ (1200 USGD/ft^2) were common.

Four clarifiers were provided - each 21 m square with 5.0 m sidewall depths. Flocculating centre wells of 8 m diameter with four flocculating mixers each were included.

SECONDARY PLANT

The layout of the plant including both primary and secondary treatment facilities is shown on Figure 3.

The secondary treatment units are as follows:

Trickling Filters:

Two filters each 23 m diameter by 7.8 m deep containing 60° cross flow plastic media. The filters are covered with fiber glass domes in order to minimize heat loss in the winter months.

Ventilation ports, sized to provide a total open area equal to 2 % of the filter cross sectional area, were provided around the base periphery. Although operators of TF plants noted the need for good ventilation, there is little data available for determination of the required ventilation area. However, one media manufacturer cited 2% as an area adequate to satisfy ventilation requirements. Normally, ventilation via convection air currents within the tower, as a result of air density differences, is adequate. Nevertheless, ducts have been provided in the bottom of the filter as a precaution in case forced air ventilation should become necessary in the future.

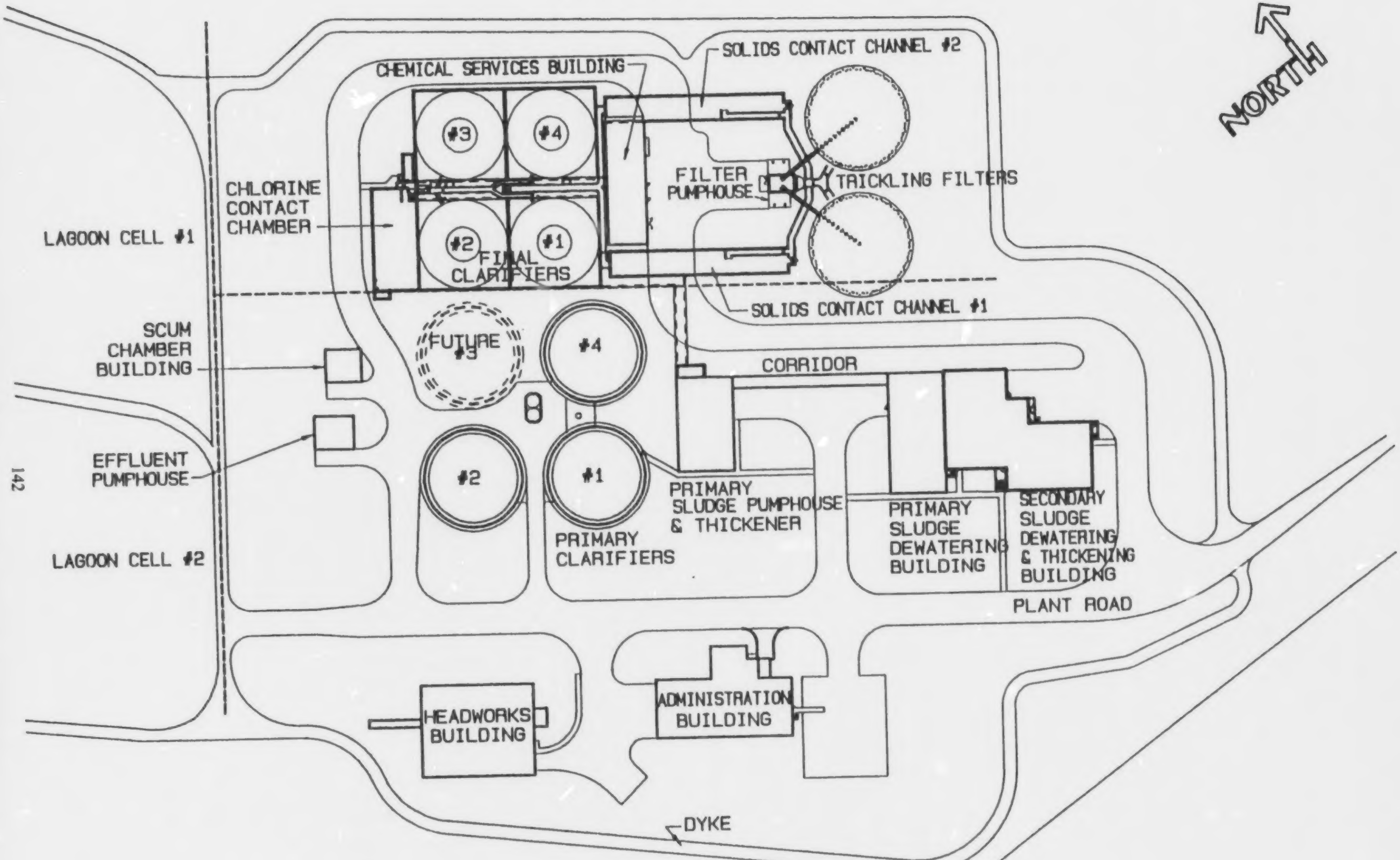
NORTH
SOLIDS CONTACT CHANNEL #2
CHEMICAL SERVICES BUILDING
#3
#4
FILTER PUMPHOUSE
TRICKLING FILTERS
CHLORINE CONTACT CHAMBER
LAGOON CELL #1
#2
#1
FINAL CLARIFIERS
SOLIDS CONTACT CHANNEL #1
SCUM CHAMBER BUILDING
FUTURE #3
#4
CORRIDOR
EFFLUENT PUMPHOUSE
#2
#1
PRIMARY SLUDGE PUMPHOUSE & THICKENER
PRIMARY SLUDGE DEWATERING BUILDING
SECONDARY SLUDGE DEWATERING & THICKENING BUILDING
PRIMARY CLARIFIERS
LAGOON CELL #2
PLANT ROAD
HEADWORKS BUILDING
ADMINISTRATION BUILDING
DYKE

FIGURE 3

Filter Pumphouse:

The pumphouse includes two duty and two standby providing 100% standby capacity. The superstructure contains a 400 kW generator capable of providing standby duty for the secondary plant.

Solids Contact Channels:

As described earlier, two SC channels have been installed providing 45 minutes retention time including recycle flow. Mixing is provided by fine bubble membrane air diffusers and two blowers - 1 duty and 1 standby. The blowers are located in the basement of the Chemical Services Building.

TF effluent can be introduced at the one third points of the channels as well as at the top end.

Final Clarifiers:

Square clarifiers were selected to conserve space and to take advantage of common wall construction. In lieu of corner sweeps on the clarifier mechanisms, concrete wedges sloping at 45° were placed in the clarifier corners.

The return secondary sludge (RSS) pumps and waste secondary sludge (WSS)pumps are contained in a gallery located between the clarifiers. The RSS is pumped to the head end of the SC channels. The WSS is pumped to the Secondary Sludge Thickening Building.

Chlorine Contact Chamber:

Retention of 20 minutes at peak flow condition is provided in the contact chamber. Mixing is provided by baffles and air diffusers. Air is provided by the same blowers used for the SC chambers.

Chemical Services Building:

This building contains the chlorine and alum feed and storage facilities. Alum dosage has varied between 70 and 130 mg/L. The chlorine dosage has varied between 6 and 8 mg/L to date.

Sludge Thickening and Dewatering:

Waste sludge, drawn from the secondary clarifiers, is thickened on two 2 m wide gravity belt thickeners. The thickened sludge is stored in aerated concrete tanks and then pumped to two 2 m wide belt filter presses for dewatering. The dewatered sludge is hauled to sanitary landfill. Filtrate from the thickeners and presses is returned to both the primary influent and the SC channels.

Effluent Pumphouse:

Treated plant effluent is pumped to the first four lagoon cells and then discharged to the river via a submerged diffuser. Dechlorination of the effluent is achieved in these cells.

PLANT START UP AND PERFORMANCE

In the first year of operation ending in March 1991, a variety of equipment problems were resolved. In general, this debugging did not significantly affect plant performance although there were occasions when problems with sludge pump control affected wasting thereby causing excessive carry over of solids. Until October 1990 plant effluent monitoring was based on samples taken from downstream of the lagoon. In October the monitoring station was relocated to the effluent pumphouse in order to assess the plant performance exclusive of the lagoons.

Limited data is available at this time to compare against the mathematical models and emperical methods used in the design. A testing program to obtain this data is planned for implementation.

To establish the best performance for the plant various modes of operation were tried. The variations revolved mostly around the RSS recycle rate, sludge wasting, and the MLSS concentration in the SC channels. The best mode of operation which evolved from the variations is one which provides a relatively low recycle rate, MLSS in the range of 1200 to 1500 mg/L and maintenance of low sludge blankets in the final clarifiers.

This mode of operation was implemented on a continuous basis starting in July this year.

In the tables below, Tables 1 and 2 show the average results for the period from January to June, 1991. In Table 3 following, the daily results from July 5 to 21, 1991 are shown.

	Influent		Primary		Final	
	BOD	SS	BOD	SS	BOD	SS
January, 1991	162.0	116.7	129.9	80.5	7.1	12.0
February	157.2	119.4	125.8	85.5	8.4	12.3
March	179.3	123.3	135.7	87.1	11.3	13.4
April	199.6	143.2	159.1	93.9	10.1	15.3
May	161.5	128.6	121.4	88.1	8.6	18.4
June	161.2	127.3	116.8	91.5	9.9	12.3
Average	170.1	126.4	131.5	87.8	9.2	14.0

TABLE 1
Plant Influent, Primary Effluent and Final Effluent BOD and SS Concentrations - mg/L

	Alum Dosage	Effluent Phosphorus
January, 1991	79.6	1.2
February	67.3	1.1
March	69.9	0.9
April	90.9	0.9
May	96.3	0.9
June	110.3	1.7

TABLE 2

ALUM DOSAGES AND EFFLUENT PHOSPHORUS CONCENTRATIONS - mg/L

Final Effluent

	BOD	SS	P
July 5	6	4	0.3
July 6	4	10	0.4
July 7	4	5	0.3
July 8	6	3	0.4
July 9	6	4	0.5
July 10	8	1	0.4
July 11	12	3	0.6
July 12	7	4	0.6
July 13	9	2	0.4
July 14	7	1	0.4
July 15	5	2	0.8
July 16	*	4	0.5
July 17		2	1.6
July 18		8	1.6
July 19		8	1.2
July 20		1	0.6
July 21		3	0.4

TABLE 3

PLANT EFFLUENT BOD, SS AND P - mg/L

* BOD results from July 15 not available at time of writing

The sludge dewatering systems have been providing the following performance:

Sludge densities:

Primary:

- from primary clarifiers - 0.5 to 1.0%
- from gravity thickeners - 6.5%
- from belt filter presses - 26%

Secondary:

- from secondary clarifiers - 1.6%
- from gravity belt thickeners - 3.5%
- from belt filter presses - 15%

DISCUSSION

BOD concentrations have been consistently well within the licence requirements and, on average, slightly below the predicted 10 mg/L averaging 9.2 mg/L for the January to June period of record.

The SS concentration for the same period averaged 14.0 mg/L. Although less than the licence requirements on average, the results were higher than the predicted 10 mg/L.

However, as shown in Table 3, since standardizing the operation to maintain low MLSS in the SC channels and a low RSS rate, the daily results for BOD and SS have been well below the predicted 10 mg/L. BOD has averaged 6.7 mg/L and SS 3.8 mg/L. It is also interesting to note that in the July 8 - 16 time frame one of the final clarifiers was out of service.

Alum dosages have been higher than the 65 mg/L predicted from lab scale testing. As noted in Table 2, dosages have varied from 67.3 to 110.3 mg/L. The high value in June is attributed to high flows and operating variables which caused some wash out of solids and corresponding high plant effluent phosphorus concentrations. It should be noted that the lagoon phosphorus effluent for all months was considerably less than the plant effluent and, for June, in particular, was 0.8 mg/L.

Similar to the situation for BOD and SS in July plant effluent phosphorus has been well below 1.0 mg/L except for July 17, 18 and 19. Alum pump problems on these days caused the higher concentrations.

The average alum dosage for the period shown in Table 3 was 85.5 mg/L. Since the licence requirement is 1.0 mg/L further fine tuning should decrease the dosage slightly.

An operating parameter that has been found useful in judging the effluent quality on an instantaneous basis has been the turbidity readings taken at the downstream end of the chlorine contact chamber. Until July an NTU reading of 10 was considered an indication of good quality effluent. Since July 5 the NTU values have been in the 2 - 4 range.

The sludge thickening and dewatering facilities have been operating reasonably well and generally as anticipated. The secondary sludge densities have, in particular, been within the design expectation of averaging 1.5%, ranging generally between 1.5 and 2.0%. It is also planned to obtain more data regarding the secondary sludge handling facilities in order to better evaluate their performance.

Data on supernatant SS in settled samples taken from the SC channels, secondary clarifier influent duct and flocculating wells indicate good flocculation of solids. The following data was obtained from grab samples:

Sample Location	Supernatant SS
SC Channel - 1/3 point	- 148 mg/L
SC Channel - 2/3 point	- 96 mg/L
Clarifier influent	- 24 mg/L
Flocculating well	- 38 mg/L

The sample taken at the flocculating well contained a relatively large number of filter flies which apparently contributed to the higher supernatant SS at this location.

Grab samples at the trickling filters indicated good reduction of total BOD through the filters. The influent total BOD was reduced from 220 mg/L to 23 mg/L.

CONCLUSIONS

During the first year of operation the plant operating procedures were varied and adjusted in order to establish optimum plant performance.

Until July, 1991 the results, on average, were reasonably good. For the January to June, 1991 period average effluent concentrations were:

BOD	-	9.2 mg/L
SS	-	14.0 mg/L
P	-	1.1 mg/L

In July the effluent concentrations improved considerably. The average results were as follows:

BOD	-	6.7 mg/L
SS	-	3.8 mg/L
P	-	0.6 mg/L

These results were obtained on the basis of operating the plant as follows:

- maintain an MLSS concentration in the range of 1200 to 1500 mg/L in the SC channels
- use a low RSS recycle rate
- maintain low sludge blankets in the final clarifier

On the basis of the data available to date, the TF/SC process is beginning to demonstrate an excellent performance record and better than had been predicted for the design.

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CAUSES AND CONTROL OF FILAMENTOUS BACTERIA IN ACTIVATED SLUDGE: MICROSCOPIC OBSERVATION NUMBER PROVIDES EARLY DETECTION OF BULKING EPISODES

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INTRODUCTION

The operator and laboratory staff of activated sludge wastewater treatment plants recognize that soluble BOD_5 removal is not a frequently observed operational problem. The overall efficiency of these plants is usually controlled by the settleability of the mixed liquor suspended solids (MLSS) or rather the separation of the MLSS from the treated liquid. The engineering significance of activated sludge was identified in 1914 (Ardern and Lockett, 1914). Albertson (1991) cited Donaldson (1932 a, b) as the first person to characterize the "weeds of activated sludge" or filamentous bacteria. The understanding of sludge bulking has progressed rapidly in the last decade. Some researchers are even developing expert systems to be utilized to mitigate bulking episodes (Chen and Koe, 1991). So far the USEPA (1987) has classified six different types of activated sludge separation problems, as follows: dispersed growth, slime bulking, pin floc, filamentous bulking, rising blanket, and foam or scum formation. The causes of filamentous bacteria proliferation and bulking mitigation will be the focus of this paper.

The main objective of the paper was to illustrate the investigative techniques used to mitigate filamentous bulking problems. The concept of the MO number was introduced and its effectiveness demonstrated.

The balance between filamentous bacteria and zoogloeal bacteria (floc-formers) is illustrated in Figure 1. Filamentous bulking will occur when the filaments extend out from the flocs in very large numbers (case I). A balance between the filaments and flocs creates ideal conditions for removing suspend solids during settling (case II). The filaments form the backbone of the flocs resulting in larger particles, which are easily settled out. Therefore occurrence of a few filaments has been identified as being beneficial to the overall separation process. The filaments may actually help filter out smaller particles, which would not have settled out normally, by physically entrapping the particles within the flocs during settling. The complete lack of filaments may lead to what is known as pin-point floc (case III). This results from the lack of a backbone for the flocs to adhere to. The net result is a low SVI, and an increase in effluent suspended solids (SSe) concentrations, which may or may not be a problem depending upon SSe concentration requirements.

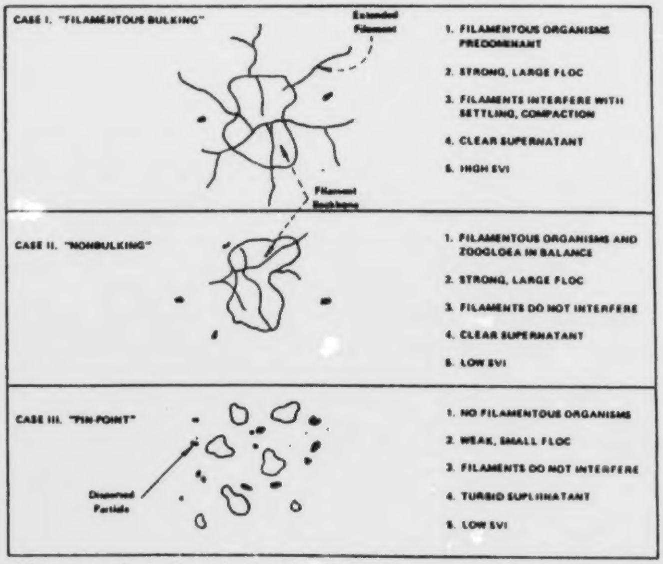

FIGURE 1 Floc development based on balance between zoogloeal bacteria and filamentous bacteria, after Jenkins, et al. (1984).

Filamentous bulking problems are not a result of a single environmental or operating condition, and the severity of bulking episodes can be variable, therefore solutions to the problems must be approached on a case-by-case basis. The design and operating strategy used for the solids separation unit operation (clarifier) can help minimize the effects of bulking. The optimization of the clarifier tank capacity should be the first consideration in either the design stages or during investigations at an operating facility. The methods of optimizing a clarifier are discussed in detail elsewhere (USEPA, 1987; WPCF, 1987; WPCF, 1985; Wahlberg and Keinath, 1988) and will not be addressed in this paper.

The practical approaches to mitigate filamentous bulking problems at operating facilities are presented in Figure 2. The short-term control measures include optimization of clarifier, return activated sludge (RAS) chlorination and polymer additions. Chlorination can be used to selectively kill filamentous bacteria in an activated sludge system. The large surface area to volume ratio of filamentous bacteria make them more susceptible then floc formers to chlorine. The bacteria exposed to the bulk liquid at the surface of a floc may also be killed by chlorination, however, the bacteria inside the floc will survive. To ensure that this is the case the dosage of chlorine must be carefully controlled. A dose which is too high may kill all the bacteria in the system including the floc formers, while a low dose may not significantly reduce the filamentous bacteria. The two most critical parameters controlling chlorine dosage are the concentration and the frequency of the dosage. It has been reported that cumulative daily dosages of 1 to 10 g CL kg^{-1} MLVSS d^{-1} were effective (Richard, 1989; USEPA; 1987). It was recommended that the cumulative dosage be spread out over a minimum of three separate applications, but success has been reported using a single daily dose (Richard, 1989). The dosages should be started low and gradually increased until effective (Richard, 1989).

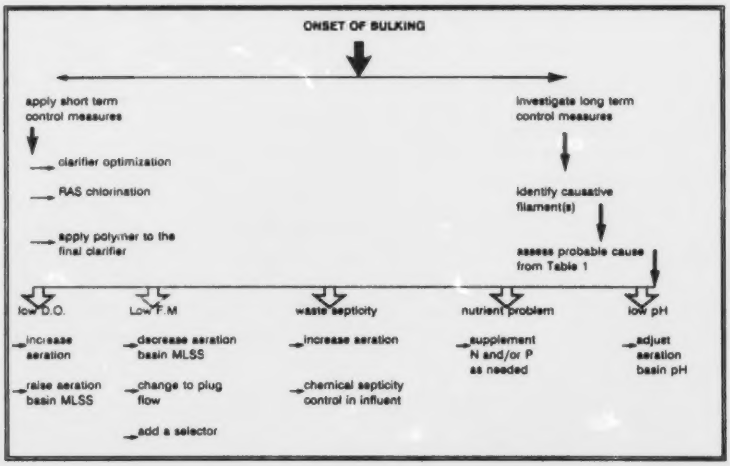
ONSET OF BULKING
apply short term control measures
clarifier optimization
RAS chlorination
apply polymer to the final clarifier
investigate long term control measures
identify causative filament(s)
assess probable cause from Table 1
low D.O.
increase aeration
raise aeration basin MLSS
Low F:M
decrease aeration basin MLSS
change to plug flow
add a selector
waste septicity
increase aeration
chemical septicity control in influent
nutrient problem
supplement N and/or P as needed
low pH
adjust aeration basin pH

FIGURE 2 Practical approaches to control bulking sludge at an operating facility, after Richard (1989).

The long-term control measures necessitate the identification of the causative filamentous bacteria (Figure 2). The identification process is conducted using a phase contrast microscope at 1000 X magnification. This technique is usually beyond the realm of normal process monitoring and would not be undertaken at the treatment plant. The details of the technique were presented in Richard (1989) and USEPA (1987). Once the causative bacteria has been identified, Table 1 can be used to review the possible environmental and operating conditions, which are supporting the bacteria. The indicative bacteria types (Table I) was compiled from a survey of 270 activated sludge plants (Richard, 1989).

The *Nocardia* species were the most commonly observed filamentous organism, but its occurrence is related to foaming, not bulking (Richard, 1989). The various practical approaches to mitigating the causative condition (Table 1) are presented in Figure 2.

TABLE 1

Dominant filament type as indicators of conditions causing activated sludge bulking.

Suggested causative condition		Indicative filament types
1.	Low dissolved oxygen (for the applied organic loading)	Type 1701, *sphaerotilus natans, Haliscomenobacter hydrossis*
2.	Low organic loading rate (low F:M)	*Microthrix parvicella, Nocardia* species, H. *hydrossis*, Types 021 N, 0041, 0675, 0092, 0581, 0961 & 0803
3.	Septic wastes/sulfides	Thiothrix species, *Beggiatoa* species Type 021N
4.	Nutrient deficiency (N and/or P)	*Thiothrix* species S. *natans* Types 021N, 0041 (industrial wastes only) & 0675
5.	Low pH (<pH 6.0)	Fungi

The traditional operating parameter used to track filamentous bulking problems is the sludge volume index (SVI) measurement (APHA, et al. 1989). The technique allows naked eye visual observations of the settling characteristics of the MLSS, as well as, the quantified SVI value, which is expressed as the liquid volume occupied by a given mass of solids. The SVI technique does not directly measure the filamentous bacteria.

A new technique allows a fast qualification of the filamentous bacteria numbers. The microscopic observation (MO) number is developed from direct microscopic observations of the MLSS at 100 X magnification (Richard, 1989; USEPA, 1987). The subjective scoring based on this method is presented in Table 2. An observer simply scores the sample based on the observations made. The technique is simple and does not require any technical skills other then the ability to operate a microscope. The drawbuck is the subjective nature of the technique, therefore care should be taken to have a single person do the observations or develop scale correction factors for various staff personnel. A typical microphotograph example of excessive filamentous bacteria is shown in Figure 3.

TABLE 2

Subjective scoring of filamentous organism abundance using the Microscopic Observation (MO) number method, after USEPA (1987) and Richard (1989).

Numerical Value	Abundance	Observation
0	none	no filaments observed
1	few	filaments observed only in an occassional floc
2	some	filaments observed in approximately half the flocs
3	common	filaments observed in all flocs but at low abundane (1-5 filaments per floc)
4	very common	filaments observed in all flocs at medium density (5-20 filaments per floc)
5	abundant	filaments observed in all flocs at high density (>20 per floc)
6	excessive	filaments present in all flocs - appears more filaments than floc and/or filaments growing in high abundance in bulk solution

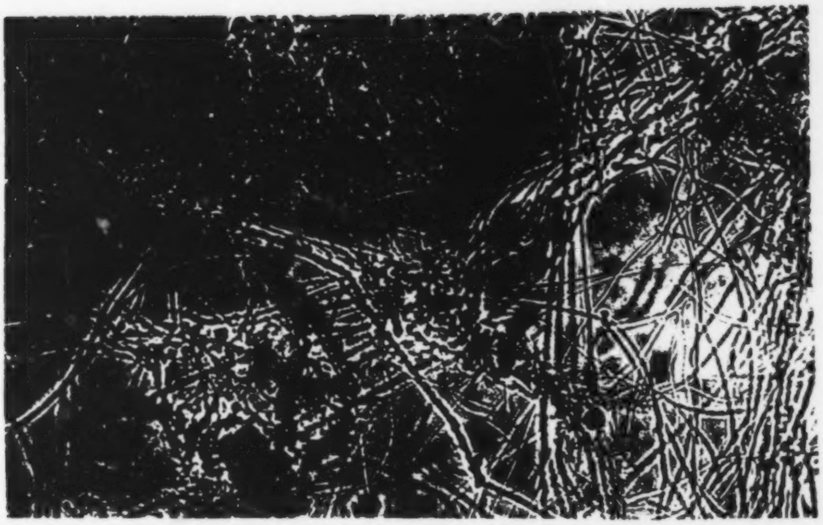

FIGURE 3: An example of excessive filementous bacteria presence, which would be scored as a number 6 (100 X magnification).

OBSERVATIONS

A small full-scale facility was experiencing filamentous bulking problems associated with the start-up of the reactor. High chlorine doses outside of the recommended range were needed to bring the bulking under control (Figure 4). The solids separation process worked efficiently if the SVI could be maintained below 300 mLg^{-1}. Chlorine was successfully used to mitigated the bulking problems during start-up and is still being used on an infrequent basis to mitigate bulking episodes.

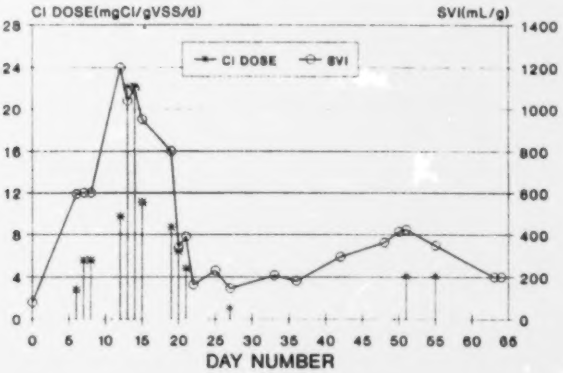

FIGURE 4 Effect of chlorine fose on SVI values at a full-scale treatment plant (100 m^3d^{-1}).

A scaled-down reactor was used to compare the SVI and MO number techniques for bulking sludge detection (Figure 5). Low doses (1.5 mg Cl g^{-1} MLVSS d^{-1}) maintained the SVI values consistent (300 mL g^{-1}) within a short time (days 12 to 15), but a longer time was needed to lower the MO number (days 12 to 26; Figure 5). On day 37 the MO number increased dramatically (MO=2.5), resulting in the chlorination of the reactor. The MO number returned to background levels (MO = 0.5) after 3 days. The SVI values did not change significantly during this episode and began to decrease to 200 mL g^{-1}. The MO number began to increase on day 43, but chlorination was not done in order to observe the response of the SVI value. The SVI value did not increase significantly until 5 days after the initial increase in MO number. In fact, clarifier failure occurred (solids carryover) a day before (day 48) a significant change in the SVI value occurred (Figure 6). This highlighted the usefulness of the MO number as an early warning technique. It predicted a sludge bulking episode 5 days before it occurred, while the SVI value did not significantly increase until it was too late to prevent the episode.

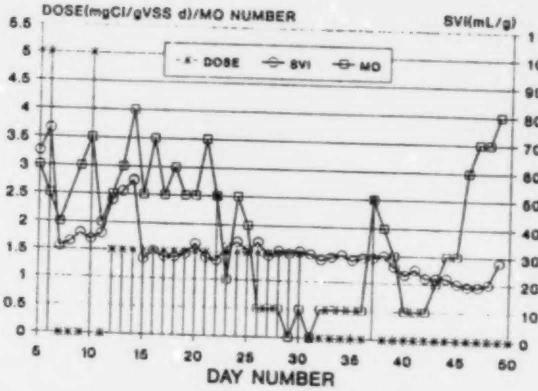

FIGURE 5 Effect of chlorine dose on SVI values and MO numbers on a scaled down facility (approximately 50 Ld^{-1}).

The effect of chlorine dosage on effluent quality is highlighted in Figure 6. A chlorine dose of 2.5 mg Cl g^{-1} VSS d^{-1} resulted in an increase in the effluent total chemical oxygen demand (CODe) and SSe on day 37. This is a result of the break-up of the bacteria cells into small particles that cannot be easily settled out. As discussed previously, the high SSe on days 48 and 49 was the result of a bulking episode and consequent solids carry-over.

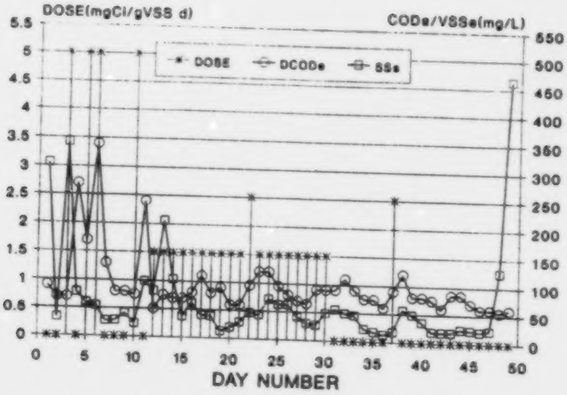

FIGURE 6 Effect of chlorine dose on effluent quality at a scaled-down facility (approximately 50 Ld^{-1}).

The most abundant filamentous bacteria identified in this sludge was type 1701. According to Table 1 the suggested causative condition was low dissolved oxygen (DO) levels, however, it was found that varying the DO from 1.5 to 8.0 mgL^{-1} did not affect the growth of the filaments. Type 1701 filamentous bacteria were reported to be fast growers (Richard, 1989; USEPA, 1987). The work of Goronszy and Eckenfelder (1986) had suggested that faster growing filaments would be susceptible to starvation during extended periods of endogenous metabolism. This could be accomplished by increasing the solids retention time (SRT) in the reactor. At higher SRTs more endogenous respiration would occur, resulting in less filamentous bacteria. The results of this investigation are presented in Figure 7. It was clearly demonstrated that an increase in the SRT resulted in a decrease in the filamentous bacteria. We must be careful, however, not to use too high an SRT. Values above 15 days were often linked to deflocculating sludge problems (caused by starving the floc formers), which leads to a cloudy effluent (McCartney, 1988).

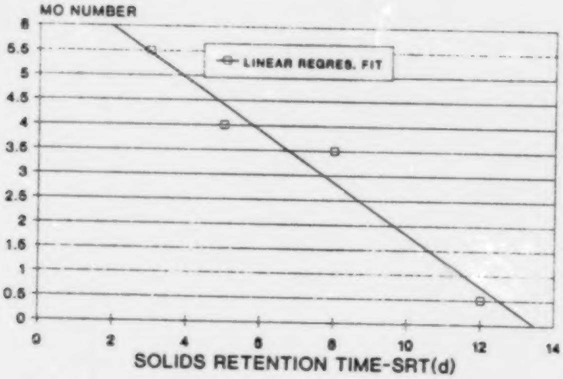

FIGURE 7 Effect of solids retention time (SRT) on the MO number of a wastewater prone to type 1701 filamentous bacteria.

CONCLUSIONS

Based on our experience the following conclusions can be made:

1) the utilization of short and long-term mitigation techniques have been successfully used to control filamentous bulking episodes,

2) the MO number technique provided an early warning (5 days) of an impending bulking episode when compared to the SVI technique, and

3) a larger SRT (increased endogenous respiration) resulted in the selective removal of the fast-growing type 1701 filamentous bacteria.

RECOMMENDATION

Based on our experience, the use of the MO number technique should be implemented at activated sludge plants prone to filamentous bulking episodes. The technique must compliment the SVI technique, not replace it.

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DISINFECTION OF WASTEWATER WITH ULTRAVIOLET LIGHT

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INTRODUCTION

In the past ten years a great deal of research has gone into the designing and testing of ultraviolet disinfection systems for wastewater. (Venosa, 1983; Thampi, 1990; Stover et al., 1986; Severin and Suidan, 1985; White et al., 1986 and Palmateer and Whitby, 1987). This has resulted in ultraviolet light being accepted as the alternative to chlorination and chlorination/dechlorination for the disinfection of wastewater.

Since ultraviolet light is not a chemical agent, no toxic residuals are produced which will affect the aquatic biota or a raw drinking water supply. This article will review the principles behind ultraviolet disinfection and the factors which affect its application to wastewater.

WHAT IS ULTRAVIOLET LIGHT AND HOW IS IT PRODUCED

The term ultraviolet (UV) light is applied to electromagnetic radiation emitted from the region of the spectrum lying beyond the visible light and before X-rays (Figure 1). The energy of a photon of light is inversely related to the wavelength; therefore UV light is very powerful compared to visible light. All UV light is invisible to the human eye. For UV disinfection, we are interested in the wavelengths between 200 and 300 nm (1nm = 10^9m).

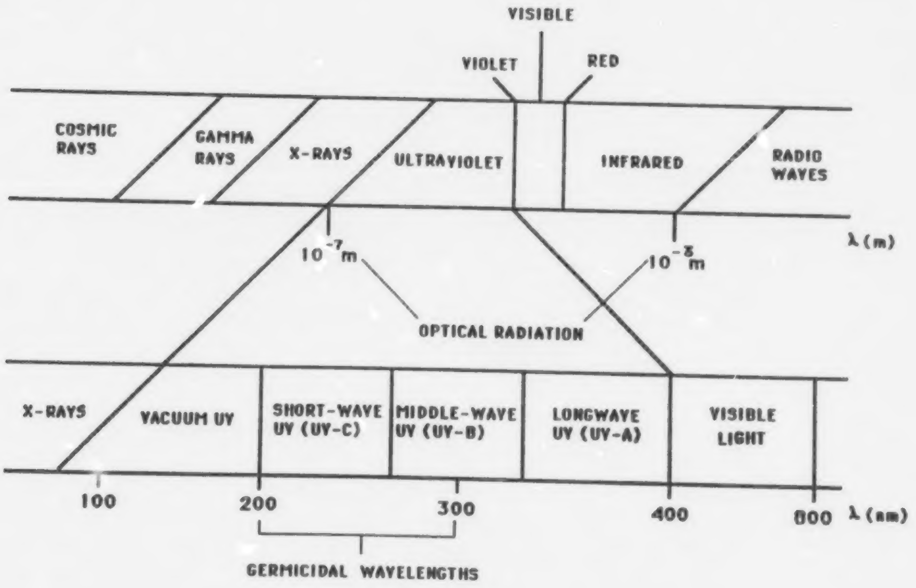
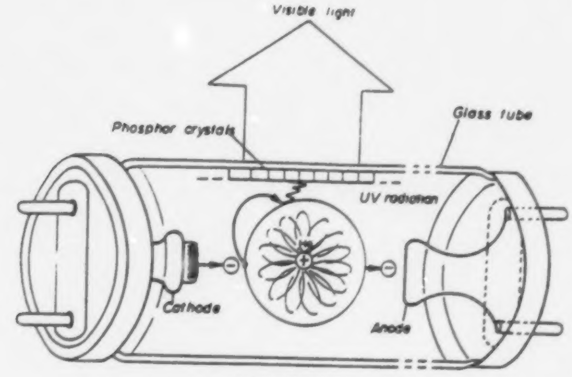
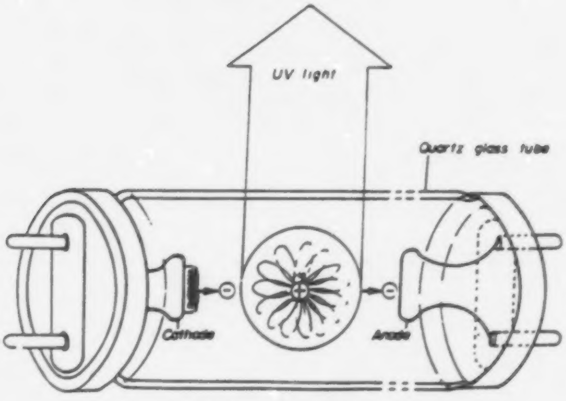


Figure 1. Electromagnetic spectrum with expanded scale of UV radiation

For practical applications, UV light must be generated from low pressure mercury germicidal lamps (Figure 2). These germicidal lamps operate electrically on the same basic principle as fluorescent lamps. UV light is emitted as a result of an electron flow through the ionized mercury vapour between the electrodes of the lamp. The two basic difference between a UV and fluorescent lamp are phosphor compounds which convert UV to visible light on the inside of the glass of the fluorescent lamp and the soft glass of the fluorescent lamp which will not transmit UV light. This soft glass and phosphor coating are replaced in the UV lamp by a highly purified form of glass called quartz which transmits ninety percent of the UV light produced by the lamp. About eight-six percent of the UV lamp's radiant energy is at a wavelength of 254nm and about one percent is at other germicidal wavelengths. Of the total power input to the UV lamp and ballast approximately 25 percent is emitted as germicidal light whereas a medium pressure lamp only produces 12 percent of its radiant energy below a wavelength of 300nm. The bluish-green glow which is observed from a UV lamp is a result of light given off by the carrier/starter gas in the lamp. This light has no germicidal action.



FLUORESCENT LAMP



GERMICIDAL LAMP

Figure 2. Comparison between a fluorescent lamp and a germicidal lamp

HOW UV LIGHT INACTIVATES MICROORGANISMS

As shown in Figure 3, UV light below a wavelength of 300 nm has the ability to kill Escherichia coli. As shown in Figure 3 the ability of UV light to destroy E. coli and the absorption spectrum of its genetic material or deoxyribonucleic acid (DNA) are identical. A microorganism can no longer divide and survive if a vital genetic function is destroyed in the DNA by the UV light. All microorganisms are destroyed by UV light. The most destructive UV light is at 260nm which is very close to the wavelength of 254nm produced by germicidal lamps. Therefore the UV light from a low pressure mercury lamp is very efficient as a germicidal agent. Figure 4 shows the log fraction survival curve of a pure culture of E. coli B/r and fecal coliforms in wastewater when they were irradiated with a constant intensity of UV light (Whitby, 1988). There is a characteristic curve like this for all microorganisms. Algae (Whitby, 1988) yeasts (Whitby, 1988) and Giardia (Rice and Hoff, 1981) are much more resistance to UV light and this results in a large shoulder or flat portion at the start of the fraction survival curve and the slope of the linear portion of the fraction survival curve is not as steep. The straight line portion on the fraction survival curve shows that a constant fraction of the number present dies in each time interval when the intensity is constant. The dose is equal to the intensity times the time (D = IT). The flattening out at the bottom of the curve of E. coli B/r occurs because some of the bacteria which are in clumps are protected from the UV light. Figure 4 also shows the effect of suspended solids on the UV disinfection of fecal coliforms. The suspended solids shield the coliforms from the UV light. The fraction survival curve flattens out at 1.5 logs higher than the laboratory culture of the more radiation resistant E. coli. Therefore, UV light is a disinfectant and not a sterilizing agent.

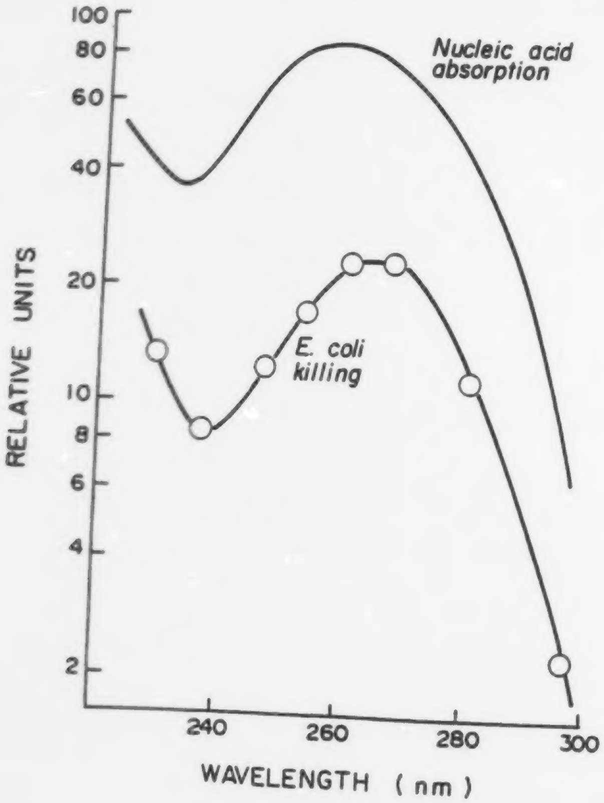


Figure 3. An inactivation curve of an UV irradiated culture of E. coli and the absorption spectrum of its genetic material.

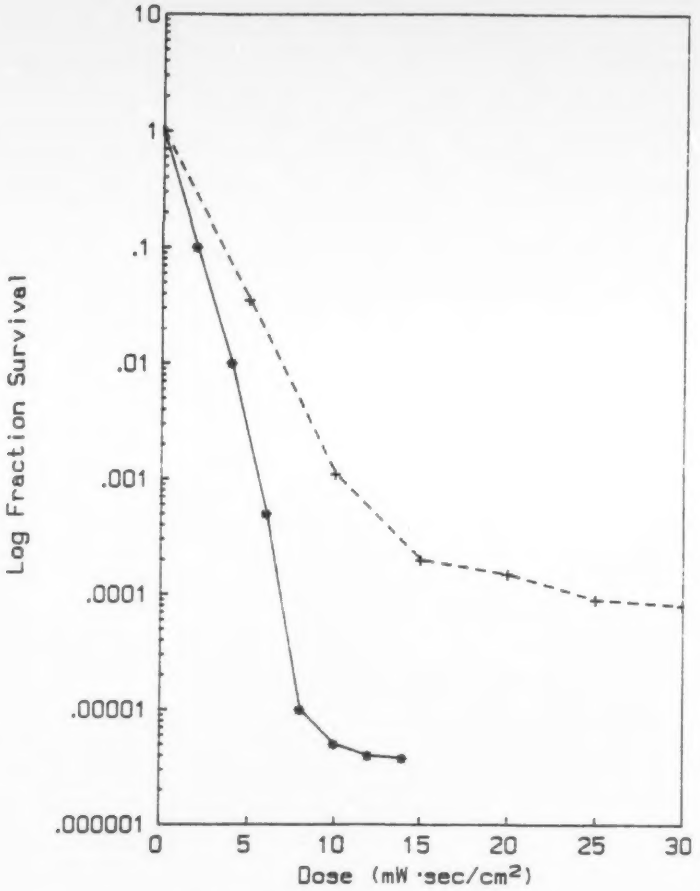


Figure 4. A comparison of the fraction survival curve of E. coil B/r (*—*) and fecal coliforms (+—+) from a wastewater treatment plant

FACTORS AFFECTING THE UV DISINFECTION OF WASTEWATER

Table 1 shows the major parameters which must be taken into consideration when a UV disinfection system is being designed for wastewater. The customer or the consultant must provide this information to the UV manufacturer because each UV system is designed on an individual basis.

TABLE 1

Factors Affecting the UV Disinfection of Wastewater

1. UV Transmission or Absorbance.
2. Suspended Solids.
3. Flow Rate or Hydraulics.
4. Iron or Hardness.
5. Wastewater Source.
6 Disinfection Requirements.
7. Lamp Life.
8. Equipment Design.

UV Transmission or Absorbance

UV lights' ability to penetrate wastewater is measured in a spectrophotometer at the same wavelength (254) that is produced by germicidal lamps. This measurement is called the Percent Transmission or Absorbance and it is a function of all the factors which absorb or reflect UV light. Figure 5 show the effect of a soluble organic compound and suspended solids on the UV transmission (Whitby et al., 1985). As the percent transmission gets lower (higher absorbance) the ability of the UV light to penetrate the wastewater and reach the target organisms decreases.

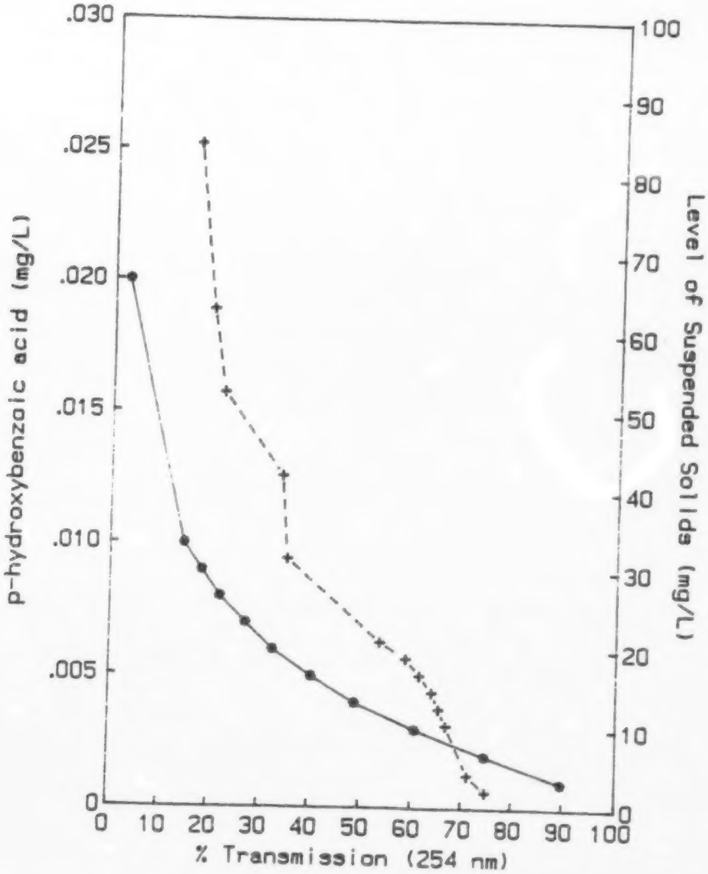


Figure 5. The effect of a soluble UV absorbing compound (p-hydroxybenzoic acid) (*——*) and suspended solids (+– – –+) on UV transmission at a wavelength of 254 nm.

Figure 6 illustrates the effect of an UV absorbing soluble compound on the disinfection ability of a parallel flow UV system with two banks of UV lamps in series. As the UV transmission decreases the number of fecal coliforms increases. Therefore the dose of UV light required is dependent upon the disinfection standard and the UV transmission. Figure 6 also shows the results of doubling the UV dose as the wastewater passes from one bank of UV lights through a second identical bank of UV lamps. By doubling the UV dose a UV transmission of 7.5 percent as compared to 24 percent can be treated to reach a fecal coliform limit of 200 per 100 millilitres. Therefore the UV system must be designed for the minimum UV transmission.

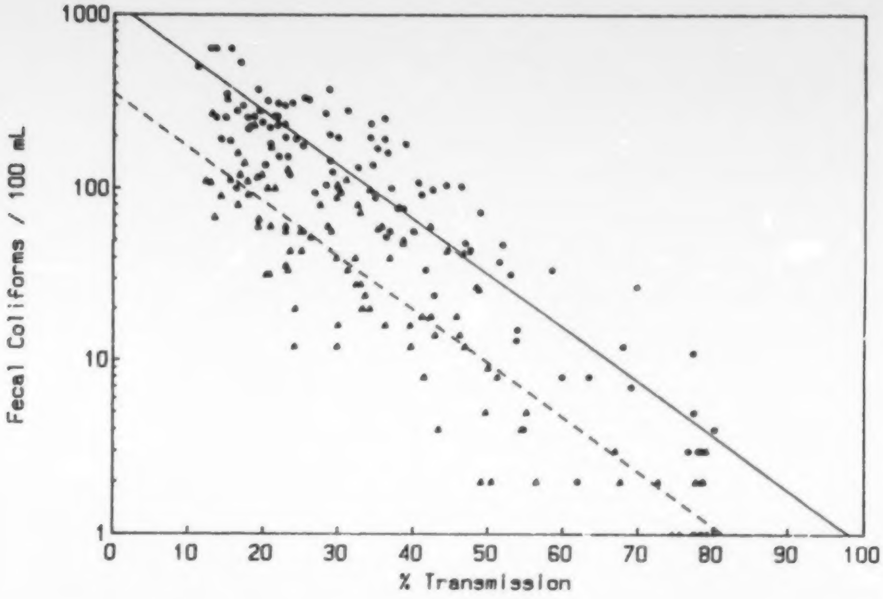


Figure 6. Effect of UV transmission on the fecal coliforms between (•——•) and after (▲---▲) two identical arrays of UV lights in a wastewater treatment plant

Suspended Solids

Some of the suspended solids in wastewater will absorb or reflect the UV light before it can penetrate the solids to kill any occluded microorganisms. UV light cannot diffuse into suspended solids with a longer contact time. If wastewaters were devoid of suspended solids UV disinfection could be used almost universally.

Figure 7 demonstrates the effect of adding suspended solids to a parallel flow UV system with two identical banks of UV lights in series (Whitby et al., 1985). As the level of suspended solids increases the number of fecal coliforms rises. This is a result of the fecal coliforms occluded in the suspended solids and being protected from the UV light. Doubling the UV dose (Figure 7) dramatically affects the level of suspended solids which can be handled.

Obtaining the proper information about the level of suspended solids is very important for the sizing of the UV system. If a wastewater treatment plant producing high levels of suspended solids is already in operation a pilot study will show the frequency of cleaning of the quartz sleeves as a result of the fouling of the quartz sleeves by the suspended solids. Pilot testing will also determine whether the fecal coliform limit can be attained.

Flow Rate or Hydraulics

The United States Environmental Protection Agency (Stover et al., 1986) provides an in-depth analysis of the effect of hydraulics on the UV disinfection of wastewater.

The number of microorganisms that are inactivated within a UV reactor is a function of the multiplication of the average intensity and residence time.

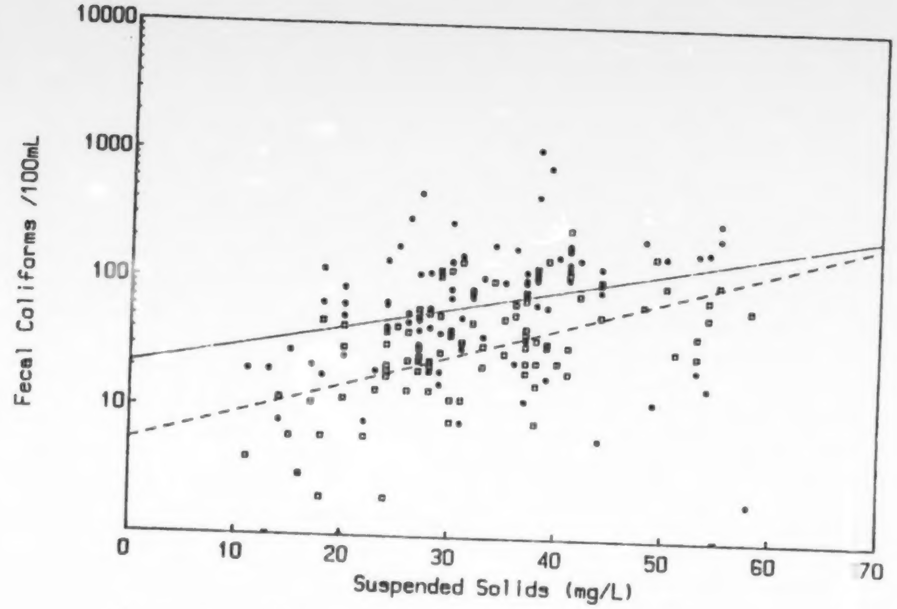


Figure 7. Effect of suspended solids on the fecal coliforms between (•——•) and after (□– – –□) two identical arrays of UV lights in a wastewater treatment plant

As the flow rate increases the number or size of the UV lamps must be proportionately increased to maintain the same disinfection requirements. Therefore the UV system must be designed for the maximum flow rate at the end of lamp life.

The UV unit must be designed so that it provides as much sideways motion as possible with very little forward mixing. This makes sure that every microorganism is exposed to the average dose of UV light. This is especially important when the water has a low UV transmission or high suspended solids. The open channel UV system where the wastewater flows parallel to the submerged lamps has a very good hydraulic profile as shown in Figure 8.

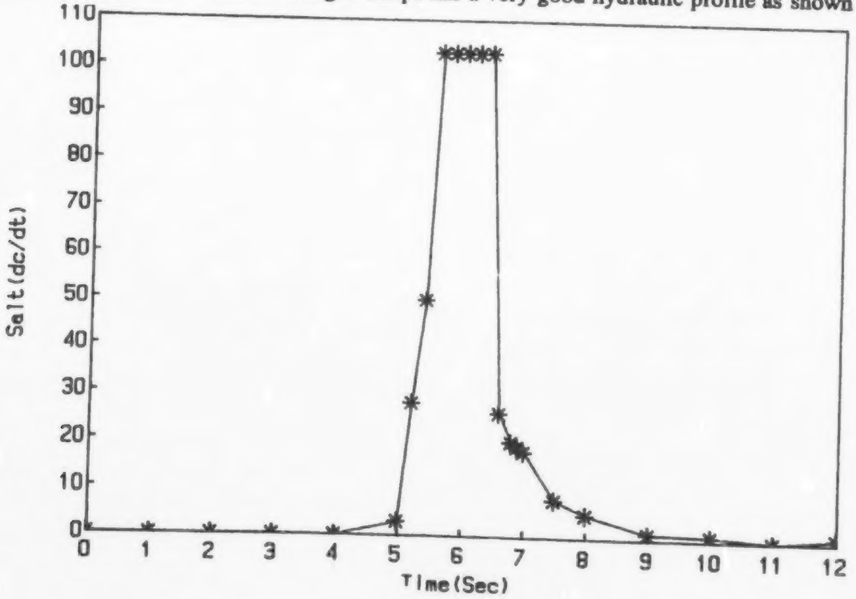


Figure 8. Hydraulic profile of an unpressurized UV system in an open channel with the UV lamps parallel to the flow of water.

The height of the wastewater above the top row of UV lamps must be rigidly controlled by a flap gate or weir at all of the flow rates. Therefore the UV system must be designed for the maximum flow rate. This is especially important if the wastewater treatment plant receives stormwater runoff.

Iron or Hardness

Iron affects UV disinfection by absorbing UV light. It does this in three ways. If the concentration of dissolved iron is high enough in the wastewater the UV light will be absorbed before it can kill any microorganisms. Iron will precipitate out on the quartz sleeves and absorb the UV light before it enters the wastewater. The third mechanism which is just being investigated is the adsorption of iron onto suspended solids, clumps of bacteria and other organic compounds. This adsorbed iron will prevent UV light from piercing the suspended solids etc. and killing the entrapped microbes. The UV industry has adopted a level of 0.3 ppm as the maximum allowable level of iron but there is no data to substantiate this limit. The level of iron should be measured in the wastewater and if it approaches 0.3 ppm a pilot study should be instituted to determine whether the disinfection level can be attained and what the cleaning frequency should be. An in-place cleaning system can be incorporated in the UV design. If possible a wastewater treatment plant should be designed with an non-iron method of precipitating phosphate.

Hardness affects UV disinfection because calcium and magnesium salts crystallize out on the quartz sleeve which surrounds the UV lamp and this prevents the UV light from entering the wastewater. A pilot study will determine the frequency of cleaning if the wasterwater is very hard so that an in-place or other cleaning system can be incorporated into the UV equipment.

Wastewater Source

It should be determined whether the wastewater treatment plant receives periodic influxes of industrial wastewater which may contain UV absorbing organic compounds, iron or hardness which may affect UV performance. These industries may be required to pre-treat their wastewater.

Disinfection Requirements

The disinfection requirements will determine the size of the UV system and whether UV light can attain the level of disinfection that is required.

As shown in Figure 4, the fecal coliforms can only be reduced to a minimum level which is characteristic for each wastewater. Below this level an increase in the UV dose has no appreciable effect.

A laboratory study using the collimated UV beam method of Qualls and Johnson (1983) will determine whether the disinfection requirements can be obtained with UV light. This data will also provide the dose of UV light which can be used to size the UV system.

Lamp Life

The manufacturers of UV low pressure mercury lamps guarantee their lamps for 7500 hours. A study by the Ontario Ministry of the Environment (Whitby et al., 1985) on a parallel flow UV system for wastewater showed that the low pressure mercury UV lamps will last much longer if the UV system is designed for the proper end of lamp life. The results of this study are shown in Figure 9. Therefore a UV system could be designed for an end of lamp life of three years. The UV lamps should only be replaced when the microbiological results become unacceptable and no other factors such as quartz sleeve fouling can be found to be causing poor disinfection.

Extending lamp life dramatically reduces the operation and maintenance cost of a UV unit.

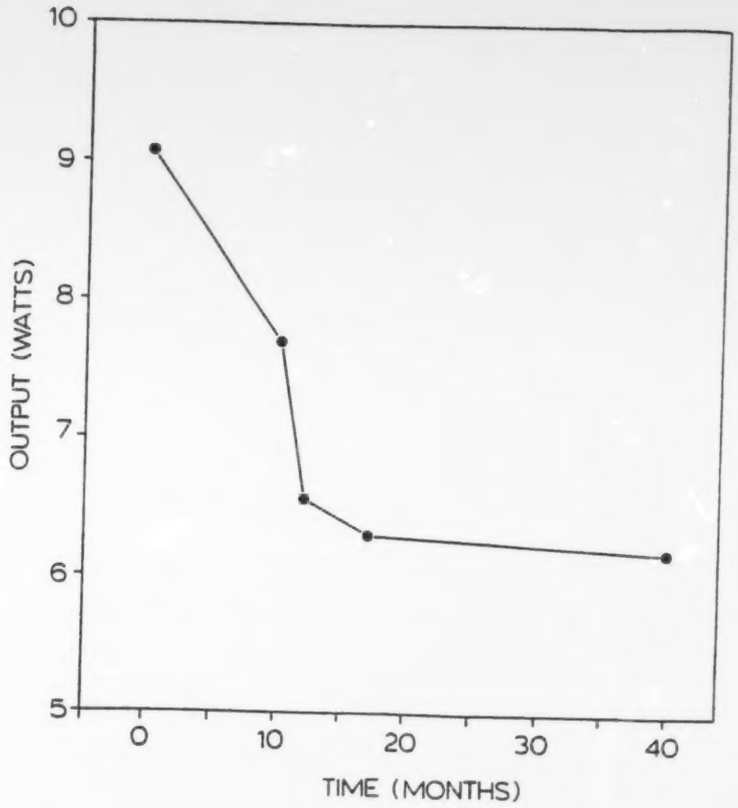


Figure 9. Results of the output of quartz - sheathed UV lamps in flowing water as determined at various lamp ages.

Equipment Design

Table 2 shows the features which should be incorporated into a UV system for wastewater.

TABLE 2

Features Which Should be Incorporated into a UV System for Wastewater

1. Total Modularity.
2. Wastewater flow parallel to the UV lamps.
3. UV lamps with electrical connections at one end.
4. Single open-ended quartz sleeves.
5. Individually isolated UV lamps.
6. UV intensity monitoring.
7. Individual lamp monitoring.
8. Hour meters.
9. Ground fault interrupters.
10. Circuit breakers, fuses.
11. Weatherproof electrical enclosures.
12. High temperature monitor and cooling.
13. Industrial multipin quick disconnect.
14. Low-water level alarm.
15. A water level controller.

The most important development in UV design for wastewater was the modular open channel system with the flow of water parallel to the submerged UV lamps. An example of this system is shown in Figure 10. Until this development pressurized UV systems for drinking water were used. These systems suffered from large pressure drops due to the accumulation of debris on the UV lamps because the water flows perpendicular to the lamps. The entire system had to be shut down to change a lamp or other component and this necessitated having one hundred percent back-up. When the water flows parallel to the UV lamps debris can only catch on the front or back of the lamp rack where it can be removed by a high pressure hose. When the UV system is modular any component can be replaced without shutting down the entire system. The open channel modular system using weatherproof components eliminates the need for pumps and buildings. A chlorine contact chamber can be turned into a UV disinfection channel.

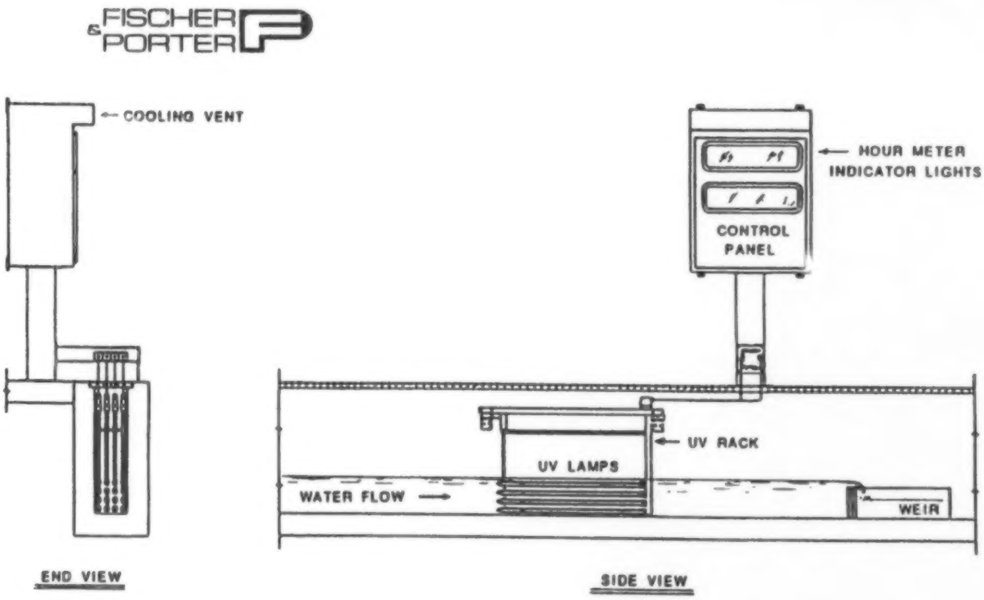


Figure 10. A schematic diagram of an open channel UV system with the UV lamps parallel to the flow of the wastewater.

Quartz sleeves which are closed at one end and UV lamps with all of the electrical connections at one end eliminates fifty percent of all the water seals and electrical connections.

Each UV lamp must be individually isolated from the rest of the system so that water cannot enter the entire UV rack if a quartz sleeve or water seal fails. The interior of the UV rack must be kept dry because the UV lamps start at 500 volts. The electrical wiring and sockets must be easily removable without soldering if a wire breaks or corrosion occurs. All of the electrical connecters should be weatherproof and easily disconnected during cold weather.

Each UV lamp must be monitored so that an indicator lights up when the UV lamp burns out. If all of the indicator lights are always on the operator must search for the indicator light which is out. When hundreds of indicator lights are involved the lights out for normal operation is a much simpler system to monitor and the indicator lights have an unlimited lifetime.

The system must have the proper ground fault detectors, fuses and circuit breakers for safe operation.

An hour meter will give the operator an indication of when to replace the UV lamps.

The control cabinets containing the ballasts for the UV lamps must be properly cooled. A high temperature monitor can be used to shut down the system if the cooling fans fail to maintain the proper temperature.

A UV sensor system consisting of an UV probe is each array of UV lamps and an alarm and/or meter will alert the operator to fouling of the quartz sleeves, aging of the UV lamps and changes in the UV transmission of the wastewater. The UV meter must only measure light at a wavelength of 254nm. Sensors using filters to eliminate visible light may be degraded by moisture and the high intensity of UV light. The operator can correlate the readings of the UV meter with the indicator organisms and determine when the quartz sleeves must be cleaned or the UV lamps must be replaced.

A low water level alarm can be used to turn off UV lights if the flow of water falls below a critical level.

A device to control the level of the water over the top row of lamps is very important because the water must be maintained between 1.3 and 2.5cm to insure proper disinfection. A flap gate or a weir can be used for this purpose.

Efficacy of UV Disinfection

Table 3 (Palmateer and Whitby, 1987) shows the results of disinfecting two parallel streams of a secondarily treated wastewater with UV light and chlorine. A free chlorine residual of 0.5 mg/L was measured at the end of a 30 minute contact time. An open channel parallel flow UV system with two banks of UV lights was used for the experiments.

TABLE 3

Microbiological Results After UV or Chlorine
Disinfection of Wastewater

		Count/100 ml	
	Influent UV	After UV	After
F-specific RNA phages	36,000	62	5,700
Somatic coliphages	1,750	<1	140
Total coliforms	108,000	130	13,600
Fecal coliforms	25,000	19	1,600
Fecal streptococci	1,800	13	725
E. coli	22,000	13	1,500

All of the microbial indicators showed that UV light was more effective than chlorine at disinfecting wastewater. Bacteriophages or viruses to E. coli were measured as a surrogate for human viruses to determine whether UV light or chlorine was more efficient at killing viruses. UV light killed significantly more viruses than chlorine. Therefore UV light may be destroying more human viruses.

SUMMARY

UV light is a viable alternative to the chemical disinfectants for destroying pathogenic microorganisms in wastewater without producing any toxic by-products. Practical experience has shown that an open channel modular UV system with submerged lamps which are parallel to the flow of water is the system of choice for ease of operation and maintenance.

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TYLEHURST SEWER RELIEF

AND POLLUTION ABATEMENT STUDIES

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INTRODUCTION

The Tylehurst Sewer District is a 236 ha catchment in the City of Winnipeg currently serviced by a combined sewer system. The Sewer District was studied in some detail in 1989 and 1990 under the City of Winnipeg Basement Flooding Relief Program. Requirements for hydraulic upgrading to meet current design standards were identified and an assessment was carried out to identify pollution abatement alternatives for mitigating pollutant loadings to the receiving course.

During the course of the study Digital Terrain Modelling (DTM) technology was integrated with the output of the Stormwater Management Model (SWMM) to produce a rather unique method of hydraulic analysis for a highly discretized computer simulation model.

Extensive use of spreadsheet software such as Lotus 1-2-3 and Quatro was also employed very effectively as an analytical tool in a number of manners. A thorough review of multiple rain gauge data versus measured flows in the conveyance system was accomplished by design of a spreadsheet utilizing the Soil Conservation Services (SCS) Unit Hydrograph Method of analysis and spreadsheet graph routines.

STUDY BACKGROUND

1) Basement Flooding Relief

The Tylehurst Combined Sewer District is one of 42 combined sewer districts in the City. Due to the impact of urbanization, changes in design standards over the years and the nature of combined sewers, the majority of the areas within the 42 districts have a considerably lower level of protection against basement flooding than the surrounding suburbs that were developed at a later date and with separate sewer systems.

Current design practice is to provide a 10 year level of protection against basement flooding to these areas (i.e. no predicted flood damage when subjected to a 10 year synthetic storm). Standard practice in the Basement Flooding Relief Program had been to upgrade service to a 5 year level of protection with relief piping followed by upgrading to a 10 year level of protection by supplemental methods such as inlet restriction.

The history of the basement flooding relief program actually dates back to the 1940's when some of the inner City catchments had the requirement for relief works identified by the Consulting Engineers, Alvord, Burdick and Howson (out of Chicago). Funding for the program was struck down a number of times by taxpayers and it wasn't until 1963 that the first inner City relief programs were finally designed and implemented (1962 just happened to be one of the most extreme summers on record with respect to high intensity rain storms). In 1963, head loss was calculated on nomographs and the "computer" to be used for high speed computation consisted of a slide rule.

Further basement flooding and the increased costs associated with damage claims eventually led the City into another major program of relief works in the 1970's and early 1980's. The introduction of computer aided analysis technology and the development of EPA's - SWMM over the 1970's led to a markedly different approach to analysis for the relief program. Existing design standards in the City were adopted during the mid 1970's which included development of the synthetic design storms currently in use. By the completion of the second major relief program, 8 of the City's 42 combined sewer districts had been upgraded to current design standards.

The Tylehurst Relief Study was part of the City's current Basement Flooding Relief Program. A 1986 report by the Waterworks, Waste and Disposal Department reviewed the remaining 34 combined sewer districts and recommended a long term program to upgrade the level of service in these areas to current design standards. As noted in their report, "An efficiently planned project priorization approach will greatly reduce the accumulated damages over the program duration." The development of calibrated computer simulation models and the use of computer technology, in general, is essential in order to meet the goals of the Flooding Relief Program. A comprehensive review of available alternatives is required to ensure that the most cost effective relief measure and implementation strategy is developed.

2) Tylehurst Combined Sewer District

The Tylehurst catchment has a drainage area of approximately 236 ha. It has the geometric shape of a long thin rectangle. It is an unusual configuration for a drainage catchment being less than 600 m wide but stretching approximately 4200 m in length from its north limit at Notre Dame Avenue to its outlet and south limit at the Assiniboine River.

Like most of the City of Winnipeg, there is very little variation in surface topography throughout the catchment. The north limit at Notre Dame is approximately the same elevation as Portage Avenue (within one block of the south limit). The elevation of all lands between Notre Dame and Portage Avenue varies less than 2 m. Lands south of Portage Avenue to the south limit drop off rapidly to the Assiniboine River.

Land use is primarily commercial and light industrial. The catchment includes such specific properties as the Polo Park Shopping Mall complex, the Winnipeg Arena and Stadium in the south, and the John Labatt's Brewery in the north. Only a small portion of the catchment south of Portage Avenue is developed as residential.

DTM AND SWMM

A critical aspect of the Flood Relief Program is benefit-cost analysis. It is the basis for priorization of the entire program as the costs of specific relief alternatives are weighed against the benefits that may be anticipated in terms of reduced flooding damages. To carry out the analysis requires a very detailed review of the hydraulic grade line (HGL) with respect to basement elevations throughout the conveyance system for numerous real and synthetic storms.

SWMM is a sophisticated mathematical model. Its output literally generates "reams" of data for each storm event. While being an exhaustive data set temporally it is almost two dimensional spatially. Its output requires considerable review to gain a picture of how the conveyance system is responding to a specific rainfall event and what this means in terms of potential flooding damage.

To gain the complete picture of conveyance system response relative to all other relevant data one may acquire in carrying out this type of study (including some irrelevant data) a data-base was assembled using the Autocad Program. The following type of information was assembled:

- The legal grid which would serve as an x, y coordinate reference for all other physical data.
- Topography of the catchment to define a z coordinate (The site was flat enough that with small exception ground surface could be defined by MH rim elevations).
- The conveyance system with x, y and z coordinates.
- Building locations were digitized in from aerial mapping which had been blown up to a 1:500 scale.
- The buildings were given a third dimension by carrying out a phone survey to assess where basements did and did not exist (being primarily commercial and light industrial land use many buildings were slab on grade construction or only had a crawl space).
- The subcatchment boundaries which had been delineated on the 1:500 aerial photography were digitized into the data base.
- All historical records on previous flooding damage claims, observed surface flooding in response to specific storms, etc.

The last piece of information to add to the data base would be the conveyance system response to rainfall events. This was achieved by creating a digital terrain model (DTM) with the ground surface primarily defined by MH rim elevations and the maximum computed HGL output from SWMM to define the conveyance system response. A contour map was then developed to define the maximum surcharge below ground level in 0.30 m intervals. A sample result is depicted in Figure 1. While the drawing is a contour map the only real points defined are at locations where the contours cross the pipe.

The resulting drawings produced can be analagous to taking a "snapshot" of the maximum HGL in a storm and being able to see in all of the conveyance system at once three dimensionally. It is a powerful method of analysis as it allows one to access and quickly review all of the pertinent information in their data base. Figure 2 is an excerpt from a June 21, 1984 simulation where the predicted response of the model can be seen to coorelate well with reported flooding damages.

The contour spacing can also provide a visual qualatative assessment of energy loss in the system. Tightly spaced contours represent a steep HGL which allows one to visually spot obviously undersized pipe elements throughout the drainage district at a glance.

USE OF SPREADSHEET ANALYSIS FOR REVIEW OF MULTIPLE RAIN GAUGE DATA

In order to develop a calibrated model it is necessary to review measured response of the conveyance system due to a known rain event versus predicted response of the model due to the same rain event. The Tylehurst Drainage District was not equipped with a rain gauge within the catchment boundaries. There were, however, 4 rain gauges within a 3.9 km radius of the catchment as noted on Figure 3.

During the 1989 monitoring period rainfall data was obtained for four well defined storm events for which measured flow data was available. The events and associated measured flows at the most downstream monitoring location are summarized in Table 1.

Table 1
1989 Rainfall Events and Measured Peak Flow

Date	Peak Flow
July 5	1.53 m³/sec (53.8 cfs)
July 12	2.36 m³/sec (83.1 cfs)
August 18 (1st storm 0230-0500)	1.27 m³/sec (44.6 cfs)
August 18 (2nd storm 1200-1330)	2.43 m³/sec (85.8 cfs)

The localized nature of convective rainfall is well documented and was well evident in the 1989 data set. Hyetographs from each of the rain gauges for each of the above noted events are presented in Figures 4 to 7.

Temporal relationships and rainfall intensities between rain gauges quite often varied far in excess of 100%. Storms were localized to the point that it would be raining at one gauge and not the other three or vice versa. Storm transposition methods are not recommended to develop a data base to calibrate SWMM, particularly with simulations of convective rainfall. However, in the absence of a rain gauge within the Tylehurst catchment some method to interpret the rainfall data was required.

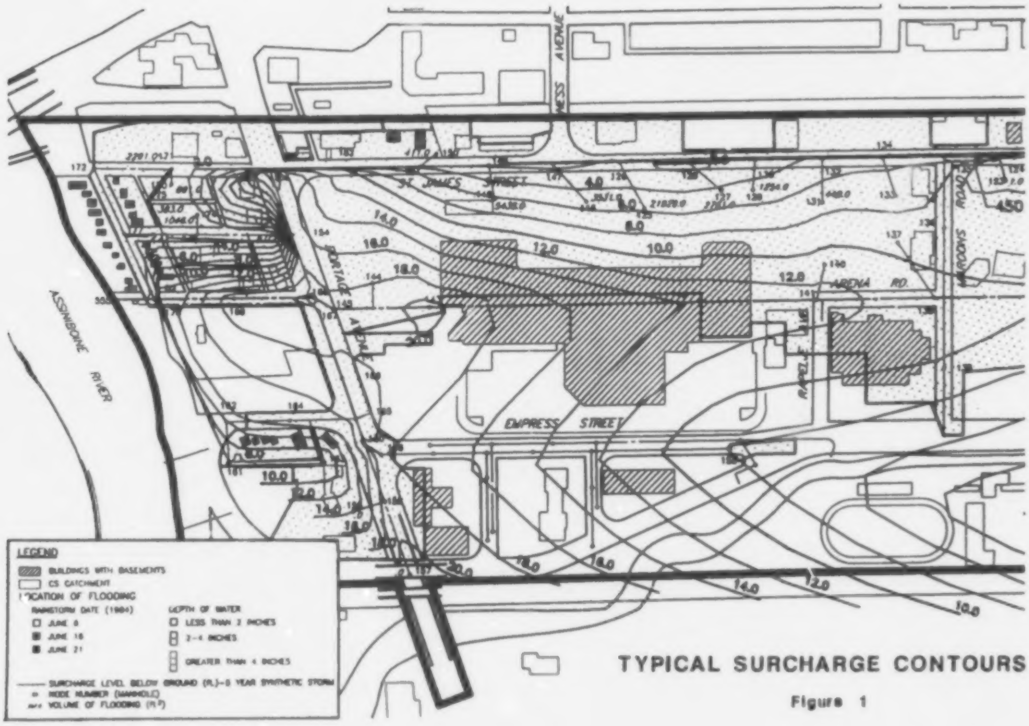
Initially, a composite hyetograph was developed similar to methods suggested by Chow for interpolation of missing rainfall records. Rainfall intensities for each time step from each station were weighted on the basis of the distance of the rain gauge to the centroid of the Tylehurst catchment. The following expression was utilized to determine rainfall intensities for each time step:

$$P_X = \frac{P_A/D_A^2 + P_B/D_B^2 + P_C/D_C^2 + P_B/D_D^2}{1/D_A^2 + 1/D_B^2 + 1/D_C^2 + 1/D_D^2}$$

Where P_X = composite rainfall intensity for Tylehurst catchment, mm/hr

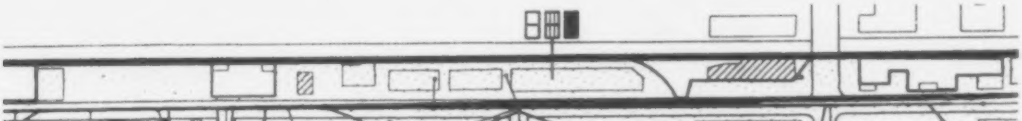
P_A, P_B, P_C, P_D = measured rainfall intensity for each gauge, mm/hr

D_A, D_B, D_C, D_D = distance of rain gauge to centroid of Tylehurst catchment



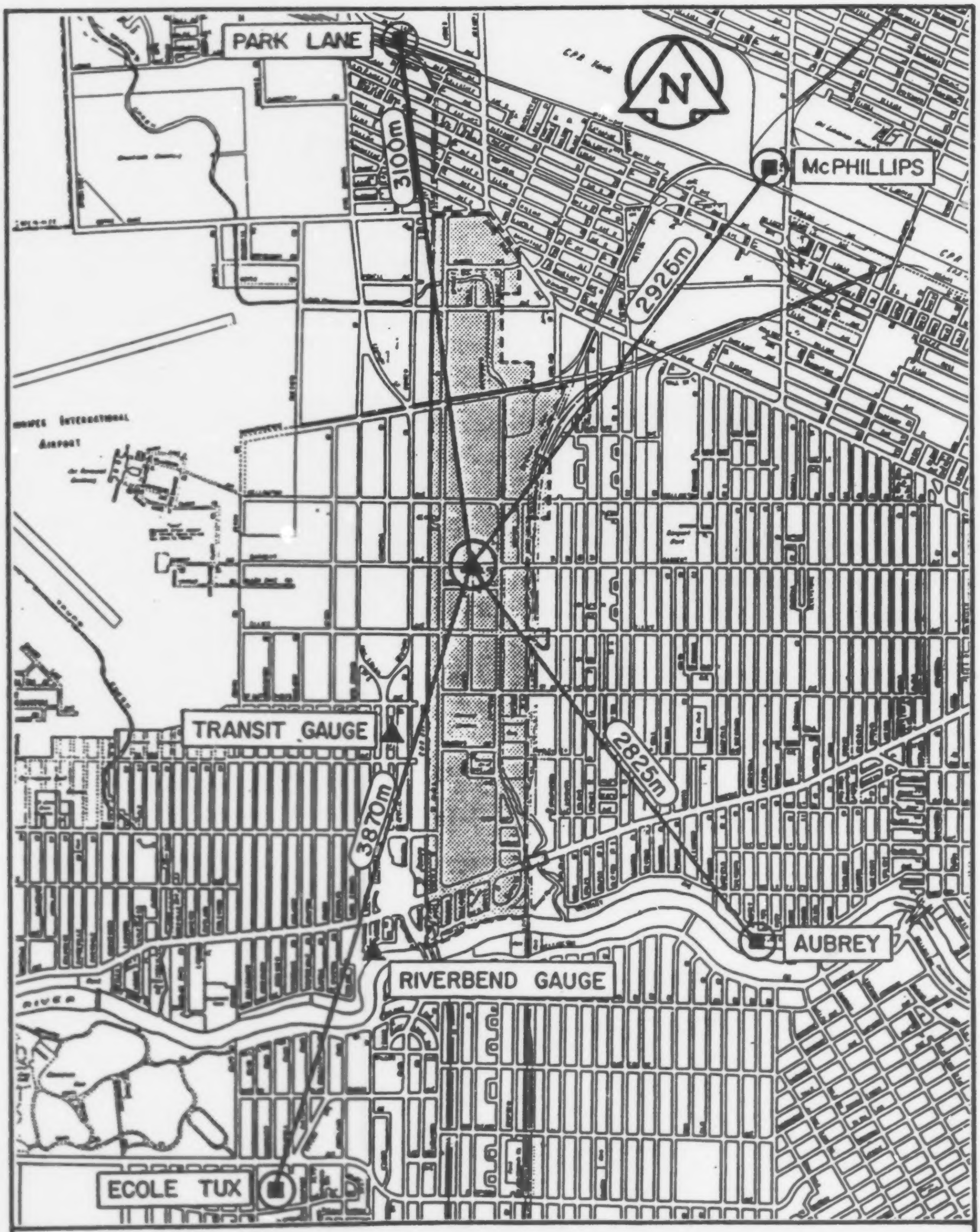
TYPICAL SURCHARGE CONTOURS

Figure 1



PREDICTED SYSTEM RESPONSE TO STORM OF JUNE 21, 1984

Figure 2



RAIN GAUGE STATIONS

Figure 3

FIGURE 4

STORM ON JULY 5, 1989

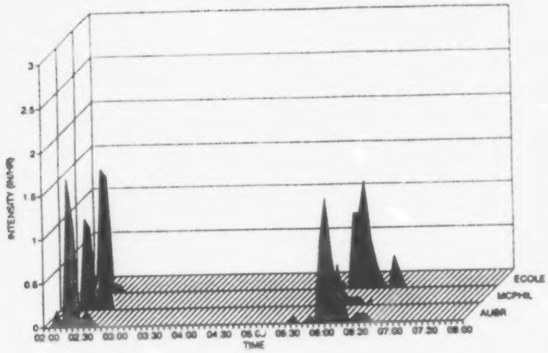


FIGURE 5

STORM ON JULY 12, 1989

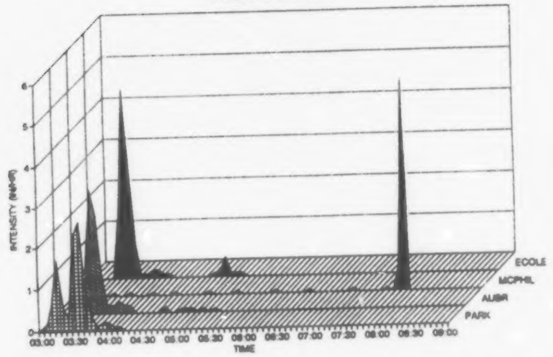


FIGURE 6

1st STORM ON AUGUST 16, 1989

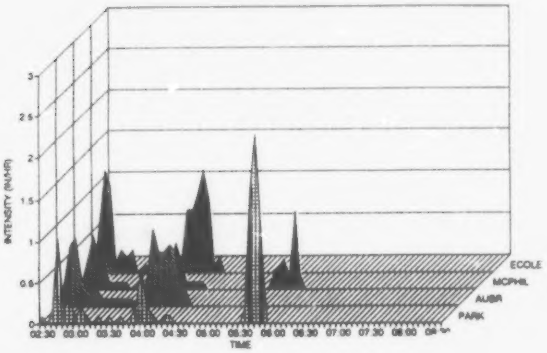


FIGURE 7

2nd STORM ON AUGUST 18, 1989

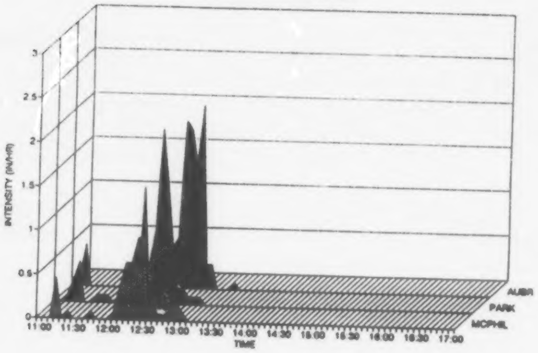


FIGURE 8

STORM JULY 5, 1989 NODE 143 (FLO-TOTE) NORTH HYETOGRAPH

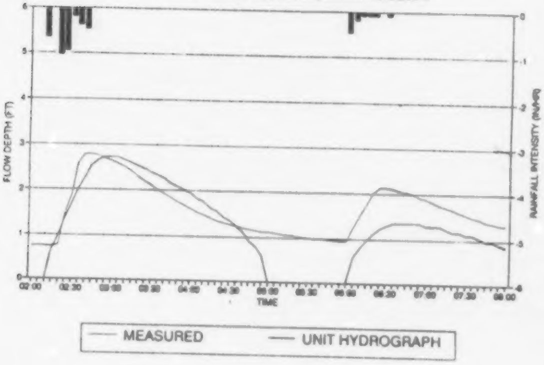
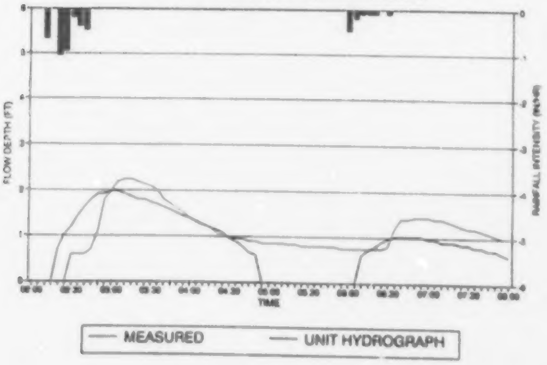


FIGURE 9

STORM JULY 5, 1989 NODE 143 (FLO-TOTE) NORTH HYETOGRAPH



This method of analysis did not produce storms that were consistent with measured system response for all storm events. This was partly due to too great a variance in measured rainfall data to determine a numerical average and the geometric shape of the catchment. The variance in rainfall data reflects the localized nature of the storm events. The geometric shape of the catchment (a long thin rectangle) makes system response very sensitive to the path of a storm over it.

While the SWMM program has the capability to input multiple rain gauges considerable time and expense would be associated with reviewing all available options. There was also the possibility that no rain gauge or combination of rain gauges was representative of the storm that passed over the catchment.

In order to develop a screening tool to quickly review what anticipated system response would be relative to individual rain gauges or combinations of rain gauges a triangular unit hydrograph method was configured on a Lotus 1-2-3 spreadsheet. The U.S. Soil Conservation Services (SCS) Unit Hydrograph Method (as presented in the MTC Drainage Manual was used to generate runoff hydrographs due to various hyetographs. The unit hydrograph method examines only the relationship between rainfall and runoff and does not include analysis of the conveyance system.

A Lotus 1-2-3 Spreadsheet was designed that allowed for review of calculated runoff hydrographs versus measured flows for the following cases:

1) A runoff hydrograph based on any one of the four rain gauge stations (4 options).

2) A runoff hydrograph based on composite data from either the two northerly stations or the two southerly stations (2 options).

3) A runoff hydrograph due to a composite of all four stations as previously noted (1 option).

Utilizing the graph function of Lotus calculated runoff hydrographs versus measured system response due to seven combinations of hydrological input could be reviewed visually and quickly for each storm event. With the spreadsheet in place each option could be reviewed literally in seconds. An assessment was then made as to which rain gauge or combination of gauges were most likely related to the measured system response. This set of hydrological data would then be utilized for input in calibration runs.

Based on the unit hydrograph analysis the following conclusions were drawn:

i) For the July 5, 1989 storm, rainfall data from the McPhillips rain gauge station alone was most representative of rainfall over the Tylehurst catchment.
ii) No single rain gauge or combination of rain gauge stations for the July 12, 1989 storm was representative of conditions in the Tylehurst catchment. Data from this storm was not considered for use in calibration.
iii) A composite hyetograph based on data from all four stations was most representative of conditions in the Tylehurst catchment for both of the storm events on August 18, 1989.

Calculated unit hydrographs versus measured flows for the monitoring points are presented in Figures 9 and 10 for the July 5, 1989 storm.

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AN OVERVIEW OF THE APPLICATIONS OF OZONE

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ABSTRACT

This paper examines the many uses to which commercially produced ozone gas may be applied. A brief review of the discovery of ozone is followed by a discussion on the mechanisms available for commercial production. Finally, the various applications for commercially produced ozone are described in terms of historical and current practice.

INTRODUCTION

The use of ozone as an oxidant and disinfectant for the treatment of drinking water is not the only use to which ozone has been successfully applied. There are numerous other applications, some directly related to drinking water treatment, others which take advantage of ozone's oxidative strength.

Ozone, what it is

Ozone is formed naturally in the atmosphere, as a colourless gas having a very pungent odour. Ozone is described as the triatomic , allotropic form of oxygen having the chemical symbol O_3 and a molecular weight of 47.9982. These three oxygen atoms form a relatively unstable, highly oxidative molecule that serves as a strong oxidant for many commercial and industrial applications. The ozone layer that is a topical subject is formed at a specific altitude, where proper conditions of temperature and pressure exist to support spontaneous photochemical reactions which favour ozone production. This article will concern itself with ozone that is generated commercially for use as an oxidant or disinfectant. The ozone formed in both cases, is of course, the same.

The distinctive "smell" of ozone can be detected by humans at levels as low as 0.01 mg/kg. It is the ozone formed by lightning discharges during a severe thunderstorm which gives the air its characteristic fresh and clean smell afterwards. Ozone is a toxic gas whose effects become more detrimental with an increase in either the time of exposure or the concentration of ozone to which an individual is exposed. Legislation in the United States has set the mean hourly standard (to be surpassed no more than once a year) at 0.1 ppm or 0.2 mg/m^3 of ozone in air. The Canadian standard is no more than 0.10 ppm during 1 hour and 0.03 ppm over 24 hours.

A history of ozone

In 1785 Van Marum noticed that air near his electrostatic machine acquired a characteristic odour when electric sparks were passed. In 1801, Cruickshank observed the same odour at the anode during electrolysis of water. Finally, in 1840 Shönbein named the substance which gave off this odour, ozone, from the Greek word "ozein" - to smell. In 1857 Werner Von Siemans designed an ozone generator that has since evolved into the present day, cylindrical dielectric type that makes up most of the commercially available ozone generators in use, and which has sometimes been called the "Siemans Type" ozone generator. The first drinking water plant to use ozone was built at Oudshoorn, Holland in 1893, while the Bon Voyage drinking water plant began operations at Nice, France, in 1906 and because it has been using ozone since that time, it is referred to as the "birthplace of ozonation for drinking water treatment". The use of ozone in the United States can be traced back to the 1940's while the first commercial scale applications of ozone in Canada took place in Quebec in 1954.

How ozone is produced

In the same way that ozone is formed naturally by the discharge of electricity during a thunderstorm, large quantities of ozone are produced commercially in the modern electrical ozone generator.

The passage of a high voltage, alternating electric discharge through a gas stream containing oxygen will result in the breakdown of the molecular oxygen, to atomic oxygen, some of the atoms of oxygen thus liberated can reform into ozone, while others simply recombine to again form oxygen. In order to control the electrical discharge, and maintain a "corona" or silent discharge in the gas space, avoiding as much as possible, arcing, a dielectric space or discharge gap is formed, using a dielectric material such as glass or ceramic. A ground electrode, constructed usually in 316L stainless steel (a material which has demonstrated high resistance to ozone oxidation) serves as the other boundary to the discharge space. This can be accomplished in many ways, but the most frequently employed geometry is that of the cylindrical dielectric (or Siemens Type) ozone generator. The cylindrical dielectric is more space efficient than other shapes and consequently more economical.

Ozone produced commercially for oxidation reactions is always produced as a gas, from air at concentrations between 1.0 and 2.0 percent by weight in air, or from oxygen at concentrations between 2.0 and 6.0 percent by weight. Since ozone is highly reactive, and has a short half life, it is very difficult to store and transport. Consequently ozone is always generated on site for immediate use.

In the cylindrical dielectric type, a glass or ceramic tube with an internal high voltage electrode, sits centred in a stainless steel, low voltage electrode. The air containing oxygen, or pure oxygen, is driven through the annular space thus formed and an electric field is applied perpendicular to the flow of gas. The ozone containing gas is collected at the other end of the cylinder for the specific process application. Variables such as voltage, frequency, gas flow, gas pressure, gas quality, gas temperature, efficiency of the generator cooling system, gas moisture content and gas oxygen content all affect the amount of ozone that will be produced. When ambient air is the feed gas, it is necessary to pretreat the air in a fashion to remove all contaminants (hydrocarbons, dust, etc.) and to dry the air to a dew point of -70°C or lower. When oxygen is used as a feedgas it is necessary to ensure an almost complete dryness as well as take the normal precautions for handling of oxygen gas.

Of the chemicals which are readily available for water treatment ozone is the most powerful, its oxidation potential being 1.5 times that of chlorine. This means that ozone will oxidize materials faster, will attain a level of disinfection faster (or at lower concentrations) and will oxidize materials that weaker oxidants cannot. ozone has sometimes been called "active oxygen" because of its very high oxidation potential ozone reacts quickly and it also decomposes quickly, reverting back to molecular oxygen. The half-life of ozone, in air, is about 20 minutes. The half life of ozone in water can be shorter than one minute or as long as 30 minutes, depending upon many factors.

In the upper atmosphere, ozone is formed as part of a complex chemical reaction that is temperature and altitude dependent. This natural process occurs all the way up through the atmosphere, and has been occurring since the formation of an oxygen containing atmosphere.

A photochemical process for which ozone has been labelled a pollutant, is another spontaneous reaction that occurs under the influence of solar radiation in the presence of nitrogen oxides and hydrocarbon pollutants emanating from the exhaust gases of motor cars. This is the situation that plagues Los Angeles and other urban centers afflicted with smog and under these circumstances ozone can be harmful to human life.

Chemically, ozone can be formed by the dissociation of oxygen rich chemical compounds, such as the catalytic dissociation of hydrogen peroxide. Chemonuclear methods can be employed, which use the large energy available from such reactions to form ozone. Another method, known as the thermal method , raises the temperature of an oxygen rich gas and ozone is formed by molecular impact.

Ozone is formed in the proximity of ultraviolet lamps and this process is employed to produce ozone commercially at low concentration levels and low production yields. Generally the UV method of ozone production does not produce ozone at a high enough concentration or in sufficient quantity to be effective.

Sizing an ozone treatment system

In all cases of water and air treatment, sizing of an ozone system is undertaken on a weight basis. The typical unit in water treatment is the weight ratio or ppm which is conveniently expressed in the metric system as mg/liter, since 1 part per million by weight equals one milligram of a chemical in a liter of water. ozone, when generated is typically defined as having been generated at a specific temperature and pressure, and is given as a weight concentration in air or oxygen, for example, 1.5%, by weight in air. This means of course, that for every 100 kilograms of air which enters the mixing device, the weight of the ozone contained therein is 1.5 kilograms, or for every 100 pounds of air which enters the mixing device, the weight of the ozone contained therein is 1.5 pounds.

When ozone is applied to a specific quantity of water, say for example a flow rate of 4 megaliters per day (˜881,000 Imperial gallons per day or 1.06 US gallons per day), and the ozone demand is determined, at say a rate of 3 mg/liter, it is readily determined that the amount of ozone required is 12.0 kg/day (or 26.46 lbs/day)

The actual determination of ozone demand is best undertaken using on site pilot equipment. If this is not possible, lab treatability studies will provide very good approximations of ozone demand and contact times required.

DRINKING WATER TREATMENT

When ozone is applied, as a gas, for drinking water treatment, it is done primarily because of its oxidative strength. This powerful oxidation potential allows ozone to be effective in the reduction or elimination of colour, taste and odour, all of which may be fundamental problems associated with a specific water supply. More importantly, ozone will effectively destroy bacteria and inactivate virus more rapidly than any other disinfectant chemical. ozone is used to oxidize heavy metals. Iron and manganese can be reduced to very low, safe levels in water supplies through ozone oxidation. This same process is used to liberate organically bound heavy metals, which are otherwise not easily decomposed by chlorine.

Ozone, applied at the start of the water treatment process, will not lead to the formation of halogenated compounds or THM's which are manufactured when chlorine is added to the raw water. Once a THM is formed, it is quite difficult, if not impossible to oxidize, even with ozone. Thus, ozone can be used as an oxidant, where it is applied at the early stages of water treatment, or it can be used as a disinfectant, where it is applied at the latter stages of water treatment. Whatever ozone gas is not consumed during the treatment of drinking water is collected and converted back to oxygen through ozone off-gas destructors, before release to the atmosphere.

There are more than two thousand installations worldwide which use ozone to treat drinking water. In France, Switzerland and Germany, ozone is the disinfection process of note. ozone is employed in most European countries. The largest ozone production facility for water treatment is located in Moscow, with a daily capacity that exceeds 20,000 lbs. ozone water treatment is used in Singapore, in Japan and Saudi Arabia.

In Canada, there are over sixty water treatment plants which use ozone in some capacity. ozone has been used extensively in drinking water treatment systems in the province of Quebec, since 1954, when the first Canadian installation was commissioned at Ste. Therese. The largest Canadian installation, at the City of Montréal, uses 3,400 kg/day (7,500 lbs/day) of ozone to disinfect 500 million gallons per day.

The first installation in Ontario, for drinking water treatment was started at the Atikokan plant in 1987 and the next Ontario potable water facility to employ ozone will be the Mannheim plant in the Regional Municipality of Waterloo, which will go online next year. Ozone is used by municipalities in Newfoundland, New Brunswick, Nova Scotia and Alberta for various water treatment applications.

Up until 1986, ozone had been installed in no more than 25 water treatment plants in the United States. With the promulgation of the EPA Safe Drinking Water Regulations of 1986, the use of ozone in the treatment of drinking water in the USA has exploded. There are presently more than one hundred and fifty ozone treatment plants under design or construction in the United States. In the next five years many hundreds of larger municipalities will be adding ozone to their water treatment plants. This list includes such cities as New York, Tucson, Phoenix, Fort Worth, San Francisco and Los Angeles. The City of Los Angeles already boasts the largest ozone system for treating drinking water in North America at a daily capacity exceeding 11,000 lbs/day. They are presently in the design stages to add as much as ten times that capacity to their total water treatment and distribution structure.

When the 1992 US EPA regulations are implemented, as many as 10,000 smaller drinking water treatment facilities will be looking to add ozone oxidation processes to their drinking water treatment.

Bottled Water

The dramatic increase in the consumption of bottled waters in North America is driven by many factors, however one element certainly has been the successful marketing of the concept of a pure alternative drinking water source. Bottled water suppliers typically seek sources of water which are free of external contamination, such as a deep aquifer and generally provide very little conventional treatment to their product.

One of the problems which the water bottler encounters is the ability to deliver a product which is both bacteria free and can offer an extended shelf life in this bacteria free condition. Many suppliers of bottled water now use ozone as the final treatment stage prior to bottling to assure that the product is free of bacteria. This is done in lieu of chlorine or the chlorine compounds since they are considered to be chemicals in the context of additives by many consumers. In fact, many bottlers are promoting on their packaging the fact that their product is "ozonated" or "ozonised" of has be "sterilized with ozone".

Typical applied dosage rates for bottled water are in the 0.1 to 1.0 mg/liter range with ozone contact times varying from four to eight minutes. The federal government, department of Health and Welfare are examining new regulations which will be applied to this industry, which will require that a 0.4 mg/liter residual be maintained in the contactor for a period of at least four minutes and that this residual be carried forward into the bottle itself.

Breweries, process & washwater

In the same way that bottled water suppliers have sought to avoid the use of chlorine as a disinfectant for their product, so too, have many breweries applied ozone to both the process and washing side of their industry.

On the production side, ozone is applied to the water as a disinfectant prior to the brewing process. Typical applied dosage rates for brewery process water, similar to the bottled water industry are in the 0.1 to 1.0 mg/liter range with ozone contact times varying from four to eight minutes.

Soft Drink Industry

The Soft Drink Bottling industry employs the same concepts as the water bottling and brewing industries when it comes to the use of sterilizing chemicals. On the production side, ozone is applied to the water as a disinfectant prior to the mixing process. Typical applied dosage rates for soft drink process water, similar to the bottled water industry are in the 0.1 to 1.0 mg/liter range with ozone contact times varying from four to eight minutes.

Distilleries

As in the brewing industry, the application of ozone to the process water offers an alternative to disinfection chemicals based upon chlorine

SWIMMING POOL WATER TREATMENT

The use of ozone in Europe, primarily in Germany, as an oxidant and disinfectant for the treatment of swimming pool water for commercial and institutional swimming pools widely practised, and there exists a detailed standard known as the DIN Standard for how swimming pool waters are to be treated, not only from the viewpoint of ozone application but from the overall process point of view as well.

Frankly, our North American experience in dealing with commercial and institutional pools and spas that are seeking alternate treatment protocols to chlorination has shown that there are a number of firms purporting to have experience in both equipment manufacture and systems applications who are misleading the potential user. Claims are being made as to process application that are not supported scientifically or by the experience of other operations. Secondly, equipment which is incapable of producing the amount of ozone, at the required concentration, under continuous operation, is being promoted by some firms, either deliberately or inadvertently. Thirdly, equipment which is incapable of providing the appropriate, efficient mass transfer and mixing of the ozone with the water to be treated is being promoted by some firms, either deliberately or inadvertently. Fourthly, the combination of the three preceding errors with unsafe practices of ozone generation and mixing are placing in jeopardy the pool operators and the bathers.

We do not wish to place a negative perspective on the use of ozone as a treatment technique for pools which, when properly undertaken, provides the best available treatment, but we further do not wish to lend our product name to an experience which will only be difficult for all concerned.

As an additional comment, the Province of British Columbia, Workers Compensation Board has developed a "Manual of Standard Practice" for the use of ozone, to which the undersigned was invited to contribute. British Columbia probably has more pools than any other Canadian province, using ozone. It also probably has more pools in Canada misusing ozone. This has led the Workers Compensation Board to produce a manual of standard practice.

The general process which we would recommend is one which has been developed during the past 20 years in the Federal Republic of Germany. In the early days of ozone treatment of swimming pool waters in Germany, many mistakes were made, and many pools had very little success using ozone. Current German swimming pool standards for treating swimming pool and hot whirlpool (spa) waters require the production of a specific water quality, and allow the use of ozone as described below, but only under specific conditions which we will point out as we discuss the treatment process. Regulations covering this German treatment process involving ozone have been adopted by the Swiss, Austrians and Belgians. The principles of ozone water treatment also have been adopted by the French, British, and other European countries.

In the United States, the first two swimming pools to install this treatment process were installed at the Karl Jewish Community Center in Milwaukee, Wisconsin in July, 1987. Operating experience to date has been very good, and the process has been approved by the Wisconsin Department of Public Health.

The proven German process, adopted by many European countries, is the basis for a Guideline for treating Public Swimming Pools With Ozone, being developed for the National Environmental Health Association. Currently, this Guideline is in the final drafting stage, and should be submitted to the NEHA for formal review in the coming months.

In North America, there is a dramatic and sustained movement to apply ozone as the final disinfectant. Again, this movement is driven by a desire to replace or supplant chlorine or its derivatives as a water treatment chemical. The North American approach has generally been characterized by a less stringent standard that that employed in Germany, primarily because North American commercial and institutional pools do not employ a flocculation/clarification stage in the process.

One application that has developed in North America is the process known as "slip streaming" or "side streaming". In this approach about 10% of the total flow is treated, on a dynamic and on-going basis, with ozone. The usual goal of this treatment is the elimination of the chloramine odour, which is formed in pools when urea and chlorine are present. ozone has been quite successful in this application.

The extension of the application of ozone to residential pools has been attempted by many suppliers promoting the use of Ultra Violet based ozone generating devices that sell in the $500.00 to $ 3,000.00 (Canadian) price range. These suppliers are generally promoting these devices, in my view, without a significant technical support either from a process point of view or from an ozone production point of view. Ultra Violet ozone generators produce ozone at concentrations of less than 0.1% by weight (when they are new) and typically do not have any air preparation process to ensure longevity of the u.v. lamp, nor consistency in the amount of ozone production.

Typical applied dosage rates for swimming pool water, similar to the bottled water industry are in the 0.1 to 1.0 mg/liter range with ozone contact times varying from four to eight minutes. The kind of pool will also influence the quantity of ozone required as will the recirculation rate of the pool. For example, a typical dosage rate for a wading pool might be 1.5 mg/liter while a wave pool would require 1.2 mg/liter.

Spa's and Hot Tubs

Because of the higher water temperatures involved some care must be given to the amount of ozone that will be used and the amount that will be effectively transferred into the water. Typical ozone dosages are in the 1.5 to 2.0 mg/liter range.

WASTEWATER TREATMENT

The treatment of wastewater with ozone, again primarily for disinfection, or at least log level of bacterial reduction was a key focus of the its use in the United States during the late 1970's and early 1980's. More than twenty plants were installed, in various locations with various design considerations. The key reason for the use of ozone was however disinfection. This approach is again being considered in light of the desire to avoid chlorination.

Effluent Disinfection

For the same reasons discussed at the outset of this paper, ozone is employed to treat effluent wastewater. When looking at waste effluents, the dosage rates required can be generally summed up as follows-

Primary Waste	20 to 100 or ,ore, mg/liter
Secondary Waste	15 to 50 mg/liter
Tertiary Waste	5 to 20 mg/liter
Advanced Treatment Effluents	2 to 10 mg/liter

Further, the degree of disinfection pursued and the degree of other competitive oxidant demands will also affect the amount of ozone required. When oxygen is already available at a facility, the use of ozone should be considered for the disinfection stage based upon the generation of ozone in that gas.

Malodorous Air Treatment

The application of ozone or ozone in combination with other chemicals for the treatment of malodorous air has a long history of success. Typically, in wastewater treatment plants, foul air can be collected and treated with ozone to reduce the odor. Ozone is usually applied in combination with wet scrubbing equipment. Typical dosage rates are in the order of 0/25 lbs of ozone per 1,000 Scfm of air to be treated.

PULP AND PAPER WASTEWATER AND PROCESS TREATMENT

As general negative reaction to the dispersion of waste effluents containing chlorinated disinfection by products grows, the consideration of the use of ozone in the many phases of the pulping process grows. It should be pointed out that the pulp and paper industry has been examining the use of ozone for many years. Ozone can be applied as a final polishing treatment to the waste effluent from the plant. Ozone is also applicable in the bleaching process and , similar to wastewater plant foul air deodorization, can be applied to the odor which emanates from the Kraft process.

INDUSTRIAL AND COMMERCIAL APPLICATIONS

Ultrapure water

The use of ozone in the printed and integrated circuit board industry for the production of ultrapure water is a well documented application. Ozone is also applied to the deionized water that is used to wash substrates during the production of printed circuit boards.

Fish Hatcheries

Using ozone instead of chlorine has some significant advantages. First, the ozone carrier gas will increase the dissolved oxygen level in the fish tanks. Second, the fish will not be exposed to the by-products of chlorination with all of the ramifications that are entailed.

Dosage rates for fish hatcheries are typically in the same order of magnitude as for low level water treatment.

Meat, Fish and Fruit Preservation

The use of ozone as a preservative during the transport of fish and vegetables has been demonstrated. A firm in the United States employs ozone, in lieu of chlorine, for the washing of broilers. There is even a process which employs ice made from ozonated water to preserve meat, fish and produce during transportation. It has been suggested that ozone ice is harder.

Kaolin Bleaching

The bleaching of kaolin clay in the United States is practised by every major producer of this material which is a brightener for paper and paints and is also used in medications.

COOLING TOWER RECIRCULATION WATER

One of the new applications to develop has bee the use of ozone for the treatment of cooling tower recirculation water both industrially and commercially in the HVAC systems. The claims for this process include the substantial reduction or even elimination of blowdown requirements thus reducing the cost of operating the facility from a water billing viewpoint. It is also argued that ozone acts as a slime and algae inhibitor. Finally ozone is said to arrest corrosion of the distribution system.

MEDICAL APPLICATIONS

Although certainly not widely practised and less widely publicized in North America, there does exist many applications of ozone in the field of medical treatment. This activity is primarily limited to Europe and most specifically to Germany. This area of ozone use is widely based upon anecdotal data and requires a significant amount of support through scientific demonstration of the various claims. I mention it in this article because it is certainly gaining prominence.

In the most simple application, using ozonated water in dental rinse systems is an indirect medical use.

The fundamental medical claims are based upon the disinfection role of ozone as a gas. It has been demonstrated that ozone as a gas, in a medical oxygen carrier, applied externally to the skin surface area around a wound will speed the healing of the wound. this claim is especially important for people afflicted with diabetes.

More drastic intervention, involving the intravenous injection of ozone containing gas or alternately the treatment of blood which is being circulated outside of the body for this purpose, is claimed by some practitioners to be a viable treatment alternative for certain viral based diseases, such as hepatitis.

It is very important to stress in this area that there are presently no medical practitioners in North America sanctioned to promote ozone therapy.

CONCLUSION

In conclusion, it is important to stress that ozone does nothing more than oxidize. That is, using a natural process, contaminants are reduced or eliminated. Because ozone is an unstable molecule, the process of oxidation is accelerated. Consequently, by generating ozone from the oxygen in the air that we breath and applying it to our water supplies, where the ozone oxidizes the contaminating compounds, we are using natures own process of disinfection. On the other hand, chlorine does two things, it not only oxidizes, less effectively than ozone, but it also chlorinates and in performing the latter it becomes in itself a pollutant of our potable water, through the formation of trihalomethanes.

Ozone is a natural oxidizing agent that performs its oxidation role more rapidly and effectively than any other oxidant or disinfectant. Ozone is very unstable, and has a very short half life, consequently, it must be produced at the site where it is to be used. Therefore, unlike chlorine, a hazardous material, which must be transported along our rail transport routes and our highways, where any kind of accident could result in the release of tons of toxic chlorine gas, ozone is used at the point of manufacture. Excess ozone is destroyed, by a simple process, and the very short half life of ozone means that it dissociates back to oxygen.

Recovery of *Giardia* Cysts Using Hollow Fiber Filtration

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INTRODUCTION

Waterborne transmission of *Giardia* is a concern to water utilities throughout North America (Davies, *et al.*, 1983; Hibler and Hancock, 1990; Lennette *et al.*, 1985; Wallis *et al.*, 1984). It is currently the most common cause of waterborne disease (Craun, 1986). Data acquired by Hibler and Hancock (1990) and LeChevallier *et al.* (1991) revealed that *Giardia* cysts are common in surface waters and can be recovered from many fully treated drinking waters. *Giardia* cysts are more resistant to disinfection than are indicator bacteria, although they can be inactivated by chlorine under appropriate conditions (Hibler *et al.*, 1987, Rice *et al.*, 1982).

Analysis of water for *Giardia* cysts is accomplished in four steps: 1) concentration, 2) purification, 3) detection, and 4) identification. The concentration of the cysts from water is typically accomplished using membrane filters (e.g., Wallis and Buchanin-Mappin, 1985) or microporous depth filters (e.g., Shaefer and Rice, 1981). In most procedures, cysts are washed out of the filter and purified using a flotation technique (e.g., Schaefer and Rice, 1981). Detection and identification are achieved using microscopic examination of the purified, concentrated sample using morphological criteria and/or fluorescent antibody staining techniques (LeChevallier *et al.*, 1990).

In general, these methods are time consuming and the recovery of cysts from large volumes of water is poor (American Public Health Association, 1989). For instance, LeChevallier *et al.* (1991) reported recovery of *Giardia* in the concentration step ranging from 26% to 88%, with an overall average of 47.2%. From this less than 50% recovery, a further 41% and 23% of cysts were lost in the purification step. As a result of the poor recovery, it is difficult to detect small numbers of cysts in finished drinking water or to determine the removal efficiency of treatment processes for these organisms.

In this study, the use of hollow fiber filter (HFF) cartridges were investigated for possible use in the purification step of the water analysis procedure. The HFF were used instead of the traditional flotation procedure to recover cysts after the concentration step and to reduce the volume of sample carried through into the detection and identification steps.

METHODS AND MATERIALS

Hollow Fiber Filter

A 50,000 molecular weight (MW) HFF (Model # HF1-43PM50, Romicon, Woburn, MA) was evaluated as a method of recovering *Giardia muris* cysts (Hyperion Research, Medicine Hat, AB) from water samples. The HFF unit consisted of many parallel hollow tubes (called hollow fibers, HF) housed in a plastic cartridge. The HFF allowed passage from the inside of the HF to the outside of the HF of substances less than 50,000 MW. Substances greater than the 50,000 MW cutoff were retained within the HF. As configured for this study, substances which were retained within the HFF constituted the recovered sample. Substances which passed through the HF were considered waste.

The HFF, depicted in Figure 1, was used as follows. The outlet port of the HFF was sealed and sample was introduced into the inlet port with a peristaltic pump. Water and particles less than 50,000 MW passed through the HF and exited through the two waste ports. The waste component was collected into a flask. The recovered sample (water and particles greater than 50,000 MW) was collected from the interior of the HF: the inlet and waste ports were sealed and the recovered sample was withdrawn through the outlet port with a vacuum pump. The recovered sample was collected into a flask. After the sample processing was complete, the HFF unit, tubing, and collection flask were rinsed with deionized laboratory grade water (LGW). The LGW was passed through the HFF using a wash bottle.

Sample Processing

All apparatus (tubing, flasks and beakers) was thoroughly rinsed with LGW prior to commencing a sample analysis. The HFF itself (see Figure 1) was cleaned by passing 2,000 mL of LGW through the unit. Both waste and recovered sample were discarded.

Four trials, called Trials A through D, were performed to evaluate the HFF for recovery of *Giardia* cysts. In the first two trials (A and B) the sample processing was carried out to maximize recovery of cysts in the sample recovery fraction and to minimize loss of cysts in the waste fraction. In Trial A, 2,150 cysts were added to 500 mL of LGW. In Trial B, 712 cysts were added to 1,000 mL of LGW. The HFF unit, tubing and collection flask were rinsed nine times with LGW using a wash bottle. The rinses were combined with the recovered sample for analysis. The waste fraction of these two trials was also saved for analysis, to see if cysts were being lost in this fraction.

The second two trials (C and D) were performed to see if the final volume of recovered sample could be substantially reduced without compromising cyst recovery. In Trial C, 3,370 cysts were added to 1,000 mL of LGW and processed as in Trials A and B. However, in this trial the rinsing of the apparatus was reduced to six times with LGW. In Trial D, 5,420 cysts were added to 1,000 mL of LGW. Sample processing was as in Trial C, with the exception that the apparatus was rinsed five times after the filtration process. The waste fractions from Trials C and D were not saved.

Sample Analysis

The recovered sample and waste fractions were analyzed for the presence of G. *muris* cysts using an indirect immunofluorescence assay (Hydrofluor-Combo, Meridian Diagnostics Inc., Cincinnati, OH). The assay involved filtering sample through a 25 mm 0.22 µm pore size cellulose acetate membrane filter (Sartorius, Gottingen, W. Germany). Thus the cysts were collected atop the membrane filter. Primary antibody reagent and labelling reagent were added to the sample, imparting fluorescence to the cysts. *Giardia* cysts on the membrane filters were identified and enumerated using an epi-fluorescence microscope (Carl Zeiss, Oberkocken, W.Germany). The manufacturer's recommended method was followed with two modifications. Tween -80 solution (0.1%) was substituted for 0.05% tween-20 solution and 2.0% 1,4-Diazabicyclo[2.2.2.] octane glycerol solution (Aldrich Chemical Co. Milwaukee, WI) was used for clearing and maintaining the fluorescence on the membrane filters. *Giardia* cysts on the membrane filters were enumerated using an epi-fluorescence microscope.

Quality Control

Two blank controls were analyzed for each trial. A blank control for the HFF and associated apparatus was prepared by pumping 500 mL of LGW through the HFF. The recovered sample was collected and analyzed. A control for the membrane filtration apparatus used in the immunofluorescence assay was prepared by rinsing the filtration apparatus with 25 mL of phosphate buffered saline (Sigma Chemical Inc.,St. Louis, MO). These blanks were used to determine if any cyst cross-contamination occurred between trials.

A *Giardia* control was also analyzed with each trial. This control sample consisted of the identical volume of cyst suspension which had been added to the LGW in each trial. The control sample was processed in parallel with the HFF but was not passed through the HFF. The number of cysts enumerated in the spike control was considered the initial concentration of cysts introduced into the HFF. This control sample also ensured that the added cysts reacted properly with the Hydrofluor-Combo reagent. To verify proper performance of the Hydrofluor-Combo reagents, a positive antigen control (supplied with the Hydrofluor-Combo Kit) was also analyzed simultaneously with the samples from each trial.

RESULTS AND DISCUSSION

In Trials A and B, cysts were enumerated in two samples: 1) the sample recovered from the HFF plus associated rinses, and 2) the waste fraction. The objective of these trials was to determine the cyst recovery capability of the

HFF. In Trial A, 2,150 cysts were added to 500 mL of water. After passage through the HFF, 2,070 cysts were enumerated in the 360 mL recovered sample and 43 cysts were enumerated in the waste fraction. As shown in Table 1, this is a recovery of 96% and a volume concentration of 1.4 times. In Trial B, the control sample indicated that 710 cysts were added to 1,000 mL of water. After passage through the HFF, 860 cysts were enumerated in the 480 mL recovered sample and no cysts were found in the waste fraction. This is a recovery of 121% (relative to the control) and a volume concentration of 2.1 times.

Trials A and B demonstrated that the HFF could yield excellent recoveries of added *Giardia* cysts when the HFF was thoroughly rinsed, i.e., without regard to final recovered sample volume. Also, cyst loss in the waste fraction was negligible.

The next part of the evaluation was to determine if the high cyst recoveries could be maintained while the final sample volume recovered was reduced. Trials C and D were conducted to investigate this aspect of the procedure using HFF.

In Trial C, 3,370 cysts were added to 1,000 mL of water. As shown in Table 1, 3,060 cysts were enumerated in 200 mL of sample recovered. This is a recovery of 91% and a volume concentration factor of 5 times. In Trial D, 5,420 cysts were added to 1,000 mL of water. In 170 mL of recovered sample, 5,650 cysts were enumerated. This is a recovery of 104% and a volume concentration of 5.9 times. Since few or no cysts had been detected in the waste fraction in Trials A and B, the waste fraction was not analyzed in Trials C and D.

Cyst recoveries, averaging 98%, remained excellent in Trials C and D. The volume concentration factor was improved from 1.4 - 2.1 times (in Trials A and B) to greater than 5 times by reducing the rinse volume. The final volume of recovered sample in Trials C and D was 200 mL or less.

The HFF was considerably easier to use than the traditional flotation procedure for purification of *Giardia* cysts. In a matter of one hour, the apparatus could be set up, sample pumped through and recovered, and apparatus rinsed. The steps of preparation of flotation solutions of precise specific gravity, flotation medium underlaying, centrifugation, cyst withdrawal and recentrifugation, necessary in the traditional flotation procedure, were eliminated. The indirect immunofluorescence assay for cyst detection and identification remained unchanged. The limitation of the HFF relative to flotation procedures for purification of cysts is that the HFF procedure removes particles smaller than the cysts while a flotation procedure removes particles heavier (larger) than the cysts. Thus, in a water sample heavily contaminated with large algal cells which could obscure the cysts in the microscopic examination, the flotation procedure would be more effective.

The HFF procedure was easily accomplished on sample volumes of 1000 mL. In very clean samples, it is estimated that 4,000 mL of original sample could be reduced to about 200 mL of recovered sample with repeated passage through the HFF. It is expected that cyst recoveries would be somewhat reduced with repeated passage through the HFF.

CONCLUSION

Recovery of *Giardia* cysts using HFF was excellent (>90% in all trials). A volume reduction factor of 5 times was easily achieved. This technique was considerably simpler and more rapidly accomplished than commonly used flotation procedures. The HFF procedure is applicable to sample volumes up to 4,000 mL and to relatively clean waters samples. The traditional flotation procedure would be the method of choice for water samples with many large particles, such as algal cells.

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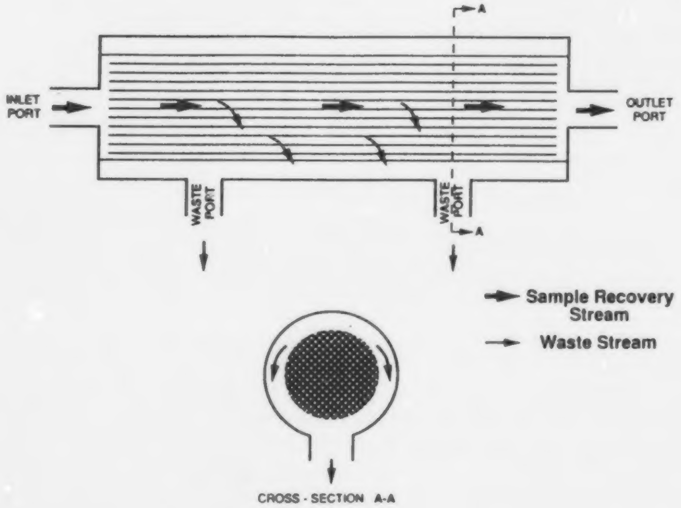
TABLE 1.

Recovery of *Giardia* cysts from hollow fiber filters.

	TRIAL A		TRIAL B		TRIAL C		TRIAL D	
	Vol (mL)	Cysts (No.)	Vol (mL)	Cysts (No.)	Vol (mL)	Cysts (No.)	Vol (mL)	Cysts (No.)
INITIAL SAMPLE	500	2150	1000	710	1000	3370	1000	5420
HFF RECOVERED SAMPLE	360	2070	480	860	200	3060	170	5650
WASTE FRACTION	700	43	1600	0	not done		not done	
TOTAL	1560	2110	3080	860	1200	3060	1170	5650
CYST RECOVERY (%)		96		121		90		104
VOLUME REDUCTION FACTOR		1.4x		2.1x		5.0x		5.9x

FIGURE 1

Schematic of hollow fiber filter.



REMOVAL OF THM PRECURSORS FROM AN AGRICULTURAL SURFACE WATER SUPPLY

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INTRODUCTION

Trihalomethanes (THMs) are chlorinated disinfection by-products which were discovered in drinking water in late 1974 (Rook, 1975). The formation of THMs is known to result from the action of chlorine on natural organics such as humic and fulvic acids. THMs are a group of halogenated organic compounds which are recognized by the US Environmental Protection Agency (USEPA) and Health and Welfare Canada (HWC) as a potential health risk. Epidemiological and case-control studies suggest an association exists between human cancer (bladder and rectal), mortality rates, and THMs in drinking water (Kanarek and Young, 1982; Gottlieb and Carr, 1982; Cantor, 1982).

In 1979, USEPA specified a maximum total trihalomethane (TTHM) level (MCL) in drinking water of 0.10 mgL^{-1} which is based on a cancer risk estimation of 100 cases per million exposed population (Reitz et al, 1982). The maximum allowable concentration (MAC) of TTHMs identified in the Guidelines for Canadian Drinking Water Quality (Health and Welfare Canada, 1987) is, however, a much higher value (0.35 mgL^{-1}). There is an indication that the MCL and MAC for THMs will be lower in the future. For example, the World Health Organization (WHO) and USEPA have recently recommended, based on additional health information, that the MCL be lowered to 0.03 mgL^{-1} in drinking water (Lykins et al, 1988).

Numerous methods have been investigated to reduce the levels of THMs in drinking water (Cotruvu 1981; Moyers and Wu, 1985). The three primary methods used in water treatment include: (1) using alternate disinfectants than chlorine (2) removing THMs after formation (3) removing the organic THM precursors. The latter method of removing THM precursors has several advantages. For example, chlorine can still be used as a disinfectant in order to maintain a residual in the system. Conventional treatment processes (coagulation, flocculation sedimentation and filtration) can be optimized. These processes can also be easily upgraded using available technologies such as preoxidation (ozone, permanganate), ion-exchange, and adsorption (powdered activated carbon, granular activated carbon) to further reduce THMs to acceptable levels.

It has been observed that agricultural surface water from drains, and algae, are major contributors of THM precursors in water reservoirs (Karimi and Singer, 1991; Amy et al, 1990; Hoehn et al, 1980). This paper summarizes the results of an experimental investigation which was conducted to determine the effectiveness of various treatment processes in removing THM precursors from an agricultural surface water (dugout) supply at the Village of Rosenort. The treatment train processes which were evaluated include preoxidation, coagulation followed by direct sand filtration, and granular activated carbon (GAC).

BACKGROUND

Case Study: Village of Rosenort

The Water treatment plant at Rosenort was constructed in 1983 and provides drinking water for the Village of Rosenort, the community of Riverside and bulk water truck hauling for various farms located in the area. The average consumption and peak demand is estimated at 227 and 511 m^3d^{-1}, respectively (Poetker, 1988).

The only source of water in the area is "spring snowmelt" from agricultural land which drains from an area of approximately 2,300 km^2 into the Morris River. An earthen reservoir with a reservoir with a storage capacity of approximately 62,000 m^3d^{-1} is periodically filled, in the fall, by pumping water from the Morris River. In the summer and fall, extreme odors and considerable algae has been observed. The reservoir is aerated to reduce taste and odor problems associated with organics in the raw water.

The Rosenort water treatment plant (WTP) rests on a reinforced concrete reservoir structure (6M x 8M). The plant (see Figure 1) was originally designed with two "greensand" filters for the removal of turbidity/color and one granular activated carbon (GAC) filter for the removal of organics to control tastes and odors. The diameters and heights of the greensand and GAC filters are 1.2 and 0.90M, respectively. Approximately 10 mgL^{-1} of a cationic coagulant (Aluminex 3) and 0.1 mgL^{-1} of potassium permanganate are added prior to sand filtration. The greensand was replaced (July 28, 1991) with silica sand due to plugging and backwash problems.

Subsequent to sand filtration (overflow rate of 2.8 $Ls^{-1}m^{-2}$), the filtered water is dropped into a ll m^3 reservoir compartment located under the plant. The sand filtered effluent is then transferred by pump into the GAC filter for the removal of organics and to control tastes and odors. The GAC filter provides an empty bed contact time (EBCT) of 3.8 minutes, at a design flow rate of 400 m^3d^{-1}. The Treated water is stored in a two compartments 226m^3 reservoir clearwell with an average chlorine detention time of 18 h. The carbon in the GAC filter was replaced on October 27, 1991 and three years ago.

TTHM levels in the consumer tapwater at Rosenort during the period from June 1985 to May 1986 are shown in Figure 2. TTHM levels reached a peak value of 0.370 mgL^{-1} during the summer and fall months and varied between 0.05 and 0.15 mgL^{-1} during the winter months (Tokarz and Belke, 1987). It is likely that high temperatures and increased algae growth were responsible for the high TTHM peak levels.

Research Objectives

The primary objective of the proposed research was to control the formation of THMs in Rosenort's drinking water. Laboratory bench scale tests were conducted to optimize the existing treatment processes at the Rosenort WTP in order to reduce the concentrations of TOC and THM precursors prior to chlorination.

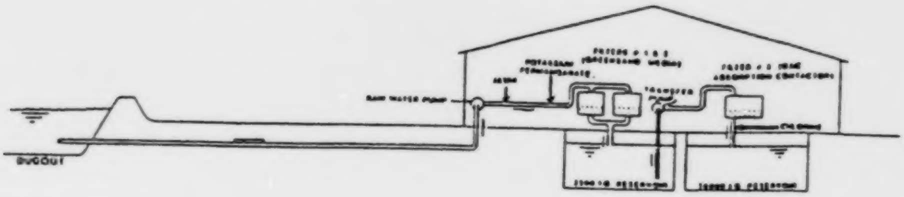
EXPERIMENTAL INVESTIGATION

General

Laboratory bench scale tests were conducted over a three month period (July 24 to Nov. 5 1991) to optimize the unit processes at the Rosenort WTP and to evaluate the efficiency of two alternative treatment trains in reducing THM precursors prior to chlorination. The experimental investigation was conducted in two runs (Run 1 & 2).

FIGURE 1

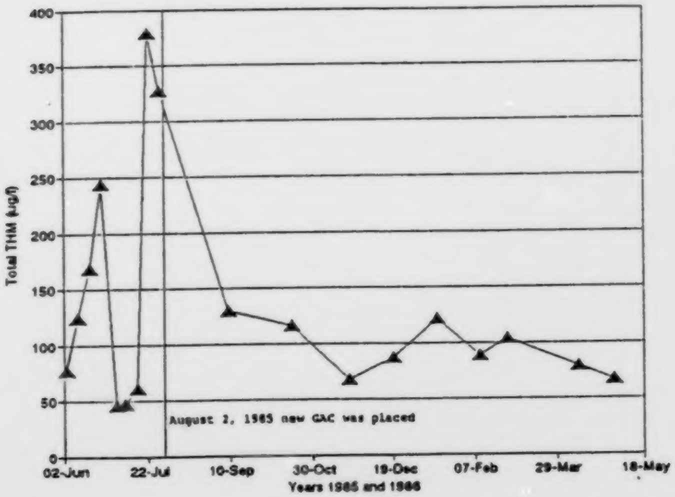
Schematic of water treatment plant at Rosenort



EXISTING TREATMENT PROCESS (40 IGPM)

FIGURE 2

TTHM levels at the Village of Rosenort (1985-1986)



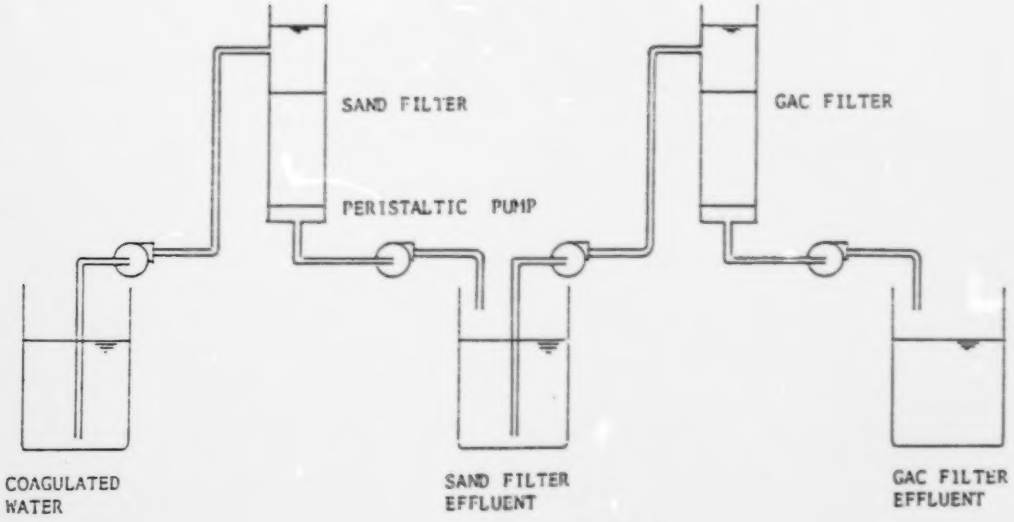
During the first run, a preliminary experiment was conducted over four weeks to evaluate the laboratory equipment and to optimize design treatment process parameters (oxidant/coagulant dosage; optimum GAC material and EBCT) for maximum removal of TOC and color. One treatment train ultilized potassium permanganate and the other treatment train utilized ozone preoxidation, each followed by coagulation using a cationic polymer, direct in-line sand filtration and GAC adsorption.

The laboratory procedures and treatment trains developed in the preliminary experiment (Run 1) were subsequently used in the second run. The final experiment, (Run 2), was conducted over a period of 8 weeks to evaluate the efficiency of each treatment train in reducing THM precursorts as measured by TOC, color, and trihalomethane formation potential.

Laboratory Procedures

Weekly samples of raw water (44L) and unchlorinated treated water (lL) were obtained from the Rosenort WTP. One liter of each sample was stored at 4° C for characterization at a later date. The remaining portion of the raw water sample was tested at room temperature conditions using the experimental set-up shown in Figure 3.

FIGURE 3
Schematic of Experimental Setup



Run 1

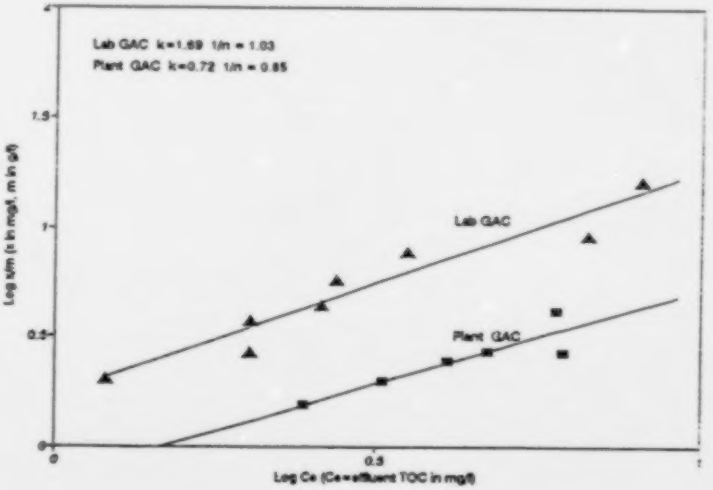
Three different coagulants were evaluated using flashmixing, followed by direct sand filtration. These include: aluminum sulfate, a blend of aluminum chloride and cationic polymer (Aluminex3) and a cationic polymer (Alchem 8103). The minimum concentration of each coagulant required to achieve acceptable levels of turbidity (1-5 NTU), color (5-15 TCU) and maximum removal of TOC were determined. Various dosages of these coagulants were added to 1.5L of raw water and flash mixed for 20 seconds using a jar test apparatus (Phipps & Bird). The coagulated water was then filtered through a small diameter (90mm) monomedia sand column at an overflow rate 1.5Ls^{-1}m^{-2}. The average effective size of the filter sand was 0.4mm with a uniformity coefficient of 1.4.

The sand filtered samples were analyzed for turbidity, color and TOC by the Visual comparison, Nephlometric and Persulfate Ultraviolet Oxidation methods, respectively (Standard Methods, 1989). The best coagulant was determined to be the cationic polymer at a dosage of 1.0 mgL^{-1}. Similar results to this polymer were obtained when using high dosages of aluminum sulfate (60 mgL^{-1}) and Aluminex3 (40 mg/L^{-1}).

An evaluation was also conducted to compare the Freundlich adsorption isotherms of the GAC media used at the Rosenort WTP to other GAC materials. It was observed (Figure 4) that the adsorptive capacity of the plant GAC (in removing TOC from the raw water) was considerably less than that of GAC (Hydrodarco 3000) material. The latter material was selected in order to optimize the removal of TOC and THM precursors.

FIGURE 4

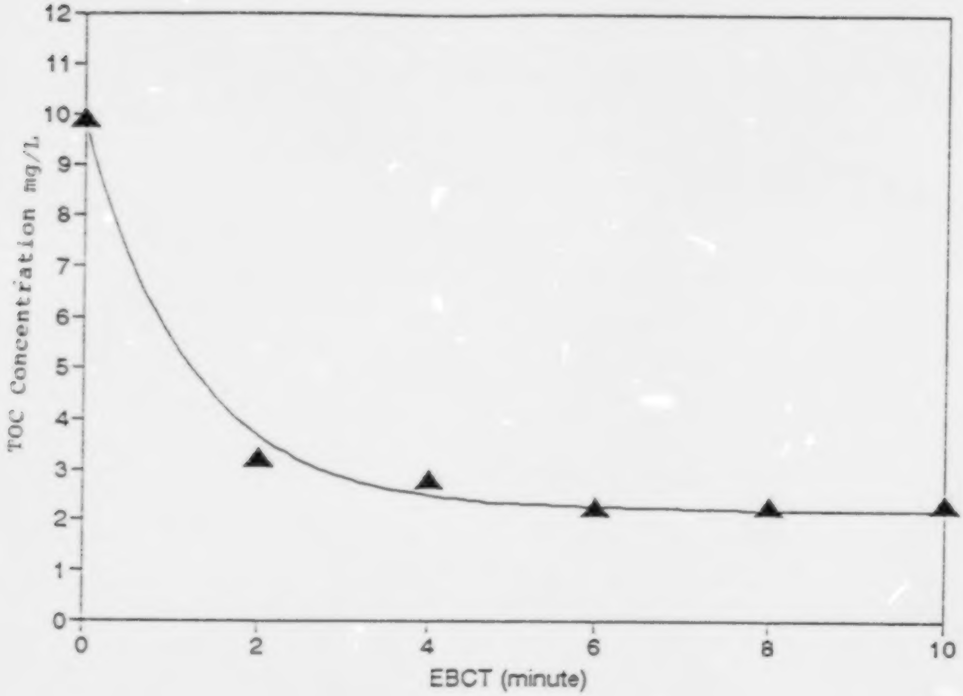
Freundlich Isotherms: Plant and Lab GAC



To determine the optimum EBCT of the Lab GAC, a sample of raw water (TOC of 10.32 mgL^{-1}) was coagulated with the cationic polymer (1.0 mgL^{-1}) and filtered through a sand column using the same procedure described above. The sand filtered effluent was then passed through a 90mm diameter Lab GAC column (volume of 1145 cm^3) to determine TOC levels at different EBCT. As shown in Figure 5, an EBCT of 4 minutes is adequate to remove TOC to a minimum value, however, a long EBCT of 10 minutes was selected to ensure effective adsorption of TH precursors. Lykins et al (1985) reported EBCT values ranging between 15 and 25 minutes for GAC contactors for larger WTP installations.

Preliminary tests (July 24-August 29) were conducted to determine the optimum dosages of potassium permanganate and ozone on the removal of TOC and color. Following preoxidation the water was coagulated with a cationic polymer (1.0 mgL^{-1}), sand filtration (1.5$L.s^{-1}m^{-2}$) and passed through a Lab GAC (EBCT 10 min) column. A treatment train not utilizing a preoxidation step was used as a control.

FIGURE 5
GAC Effluent TOC versus EBCT



Two dosages of potassium permangnate (1.0 and 1.5 mgL^{-1}) were added to 1.5L of raw water sample, and flashmixed for 20 seconds using a jar test apparatus (Phipps & Bird). The oxidation detention time (ODT) prior to coagulation was set at 1 h. The selection of the dosages and ODT were based on the results of an experimental study by Gifford et al (1989) who showed, with 1 h contact time, that a maximum concentration of 1.0 mgL^{-1} could be used without exceeding acceptable levels of the residual permanganate ion (0 05 mgL-1) concentration in the treated water. The study also showed that a dosage of 1.0 mgL^{-1} also reduced THM precursors by 30%. In this work, the concentration of the permanganate residual was measured using the titrimetric phenylarsene oxide (PAO) method developed by Beck (1968).

Ozonated waters of four different concentrations (0.7, 2.0, 5.0, and 10.0 mgL$^{-1)}$ were prepared from the raw water samples using the apparatus shown in Figure 6. The apparatus consisting of an ozonator (Ozonair), ozone contactor (2.5L) and a series of bottles (2.5L). The concentration of ozone was measured by the Potassium Iodide Method (Standard Methods, 1989). Samples of raw water (control), permanganated, and ozonated waters were flash mixed with a cationic polymer (1.0 mgL^{-1}), and filtered through the sand and Lab GAC column.

The preliminary test results showed that the optimum dosage for permaganate and ozone oxidation was 1.5 and 2.0 mgL^{-1}, respectively. A higher permanganate dosage was selected (e.g. Rosenort WTP: 0.1 mgL^{-1}) to optimize the removal of THM precursors. An ozone dosage of 2.0 mgL^{-1} resulted in the maximum removal of TOC. A summary of the optimum design control parameters for each treatment train is shown in Table 1.

FIGURE 6
Ozonation Apparatus

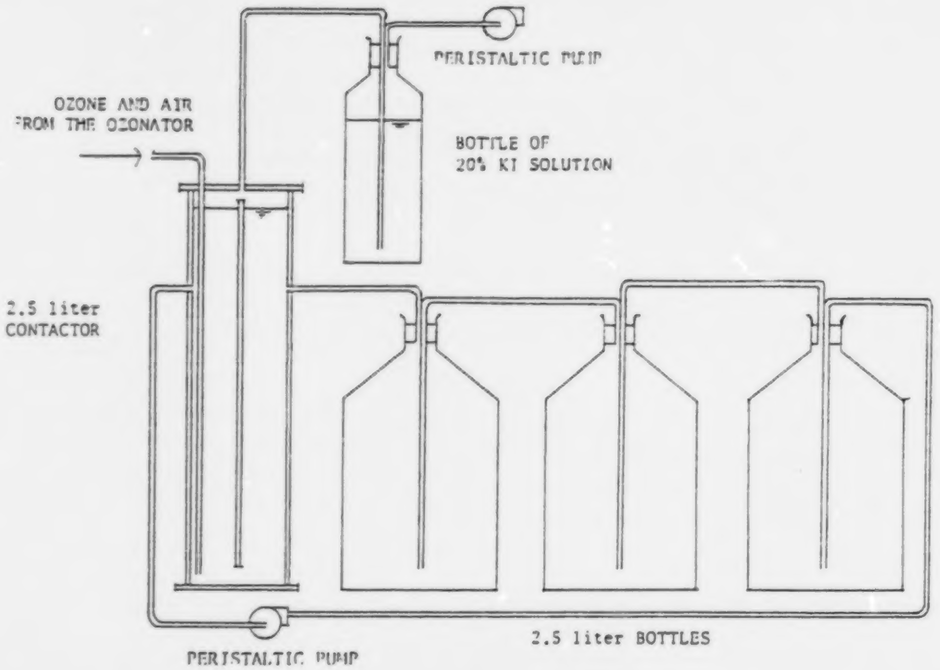


TABLE 1
Optimum Design Control Parameters

Treatment Train	Oxidation Step	Coagulant Dosage	Sand Column	Lab GAC Column
A	Potassium Permanganate (1.5 mgL^{-1})	Cationic Polymer (1.0 mgL^{-1})	Overflow Rate (1.5L.s^{-1}m^{-2})	EBCT 10 min
B	Ozonated Water (2.0 mgL^{-1})	Cationic Polymer (1.0 mgL^{-1})	Overflow Rate (1.5L.s^{-1}m^{-2})	EBCT 10 min

Run 2

The final (9 week) experiment was based on the treatment trains (A&B) shown in Table 1. The efficiency of each treatment train (A&B) in removing THM precursors was measured by the reduction in TOC and color and compared to a control treatment train (non-oxidized water, coagulation, sand and GAC filtration). The same laboratory test apparatus and procedures were used, however, the efficiency of each unit processes with respect to THM precursor removal were also measured in terms of trihalomethane (chloroform) formation potential (THMFP).

The THFP of a water is an index of the potential extent of THM formation after the application of chlorine. Four THMs which are commonly found in drinking water include: chloroform ($CHCl_3$), dichlorobromomethane ($CHCl_2Br$), dibromochloromethane 9$CHClBr_2$) and bromoform ($CHBr_3$). Before comparisons can be made between surrogate parameters (TOC, Color), a decision was made as to how THMFP should be measured.

Bull (1982) showed that chloroform accounted for more than 75% of the maximum TTHM levels found in chlorinated tapwater. Thus a decision was made in this work, based on economic considerations, to measure THMFP concentrations in terms of only chloroform. The primary objective of conducting the THMFP tests to determine the efficiencies of the different unit processes with respect the THM precursor removal. Test conditions used in this experiment (e.g. as incubation time, chlorine, dosage, pH, temperature) are described to allow comparisons to be made to other studies (Tan and Amy, 1991).

Raw and treated water samples were collected during the last two weeks of the final experiment and analyzed to determine the potential for chloroform potential after sand and GAC filtration . The THMFP samples were buffered with borate and adjusted to a pH of 8.2 by the addition of either sodium hydroxide or hydrochloric acid. The adjusted samples were transferred into (3) 300 mL BOD bottles and spiked with sodium hypochlorite (equivalent of 3 mg Cl_2 per 1mg TOC) unit process. Stoppers were placed and incubated at 7 days at 30°C. Following the incubation period, the samples were transferred into 40mL born-glass screw teflon septum vials which contained sodium thiosulfate (reducing agent) granules. These vials were sent to an independent laboratory (W.M. Ward Technical Service) for measurement of chloroform by GC analysis (Standard Methods, 1989).

Other test parameters which were monitored include pH, turbidity, alkalinity, total hardness, calcium hardness, and chlorine residual. These parameters were measured in accordance with Standard Methods (1989). The concentration of the permanganate residual was also measured using the PAO titrimetric method developed by Beck (1968).

RESULTS AND DISCUSSION

The characteristics of the raw and unchlorinated finished treated water at the Rosenort WTP (Sept.6-Nov.11), are shown in Table 2. The quality of the treated water is good in terms of acceptable turbidity and color levels. It is evident that the alkalinity and hardness levels did not change during treatment. The concentration of permanganate in the treated water (0.,008 mgL^{-1}) is within accptable limits. The concentration of THMFP in the raw water (1.118 mgL^{-1}) as $CHCl_3$) the THMFP is surprisingly very high. For example, the THMFP in the City of Winnipeg raw water at Shoal Lake was measured to be 0.770 mgL^{-1} as $CHCl_3$. The efficiency of the Rosenort WTP in removing THM precursors is only 30 percent which results in a high level of THMFP in the treated water (750 mgL_{-1} as $CHCl_3$).

A typical plot of the concentration of TOC versus time for the control (non-oxidized) treated water samples taken after coagulation/sand filtration after coagulation/sand/GAC filtration is shown in Figure 7. It is evident that the removal of TOC following GAC filtration is much higher. For example, the average TOC of the raw water samples was reduced from 11.0 to 9.6 mgL^{-1}. The results show that coagulation followed by sand and GAC filtration reduced TOC levels by 10.9 and 88.2 percent, respectively.

TABLE 2
Rosenort WTP: Raw and Finished Water

Test Parameter	Raw Water			Finished Water		
	n	Average	Standard Deviation	n	Average	Standard Deviations
TOC (mgL^{-1})	9	11.0	1.5	9	9.6	(0-9)
Color TCO	9	22	6	9	8	2
THMFP mgL^{-1} $CHCL_3$	2	1.118	0.25	1	0.785	-
PH	9	8.2	0.2	9	8.1	0.2
Turbidity NTU	6	14.5	7.0	6	1.1	0.8
Alkalinity (mgL^{-1})	9	196	7	9	195	8
Total Hardness mgL^{-1} as $CaCO_3$	9	271	18	9	268	19
Calcium Hardness mgL^{-1} as $CaCO_3$	9	160	9	9	156	9
Permanganate (mgL^{-1})	9	0.065	0.020	9	0.008	0.001

FIGURE 7
TOC Versus Time: Control (Non-oxidized) Treatment

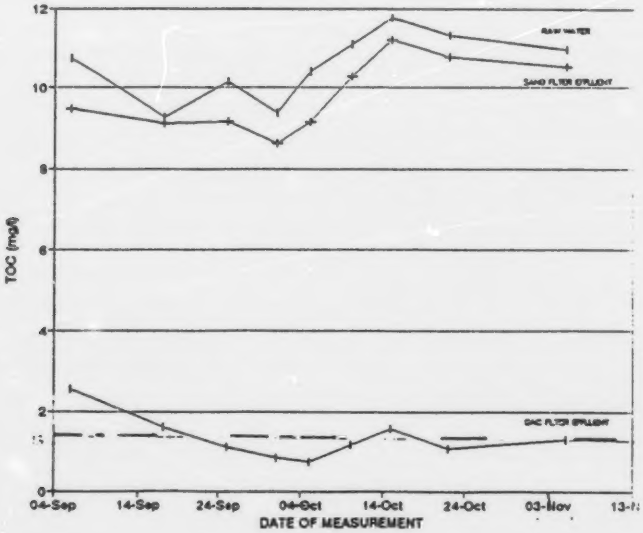


TABLE 3
Final Experiment: Test Results

Test Parameter	Control (non-oxidize)		Treatment A (Permanganate)		Treatment B (ozone)	
Treated Sample	Sand filtered	GAC filtered	Sand filtered	GAC filtered	Sand filtered	GAC filtered
TOC mgL^{-1}	9.8 (0.9)*	1.3 (0.5)	9.6 (1.0)	1.1 (0.4)	9.9 (0.9)	1.4 (0.5)
Color TCV	16 (2)	< 5	28 (10)	14 (11)	9 (2)	<5
THMFP mgL^{-1} $CHCl_3$	1.090 (.014)	0.047 (.007)	1.229 (.100)	0.047 (.011)	1.125 (.035)	0.041 (.011)
pH	8.2 (0.2)	7.4 (0.2)	8.3 (0.1)	6.9 (.3)	8.3 (0.1)	7.2 (0.2)
Turbidity NTU	1.5 (0.5)	1.2 (0.2)	2.3 (0.7)	1.4 (0.3)	1.0 (.4)	0.9 (.2)
Alkalinity mgL^{-1}	200 (6)	187 (9)	200 (6)	149 (25)	200 (2)	174 (13)
Total Hardness mgL^{-1}as $CACO_3$	270 (17)	253 (26)	270 (18)	241 (29)	271 (17)	246 (29)
Calcium Hardness mgL^{-1} as $CaCO_3$	163 (8)	149 (16)	161 (12)	140 (13)	161 (10)	144 (16)
Permanganate mgL^{-1}	0.040 (.007)	0.04 (.004)	0.187 (.004)	0.154 (.003)	0.049 (.009)	0.040 (.006)

Brackets - standard deviation

Table 4
Average Percent Reduction

Test Parameter	Control (non-oxidize)		Treatment A (Permanganate)		Treatment B (ozone)	
Treated Sample	Sand filtered	GAC filtered	Sand filtered	GAC filtered	Sand filtered	GAC filtered
	%	%	%	%	%	%
TOC mgL^{-1}	10.9	88.2	12.7	90.0	10.0	87.3
THMFP mgL^{-1} $CHCl_3$	2.5	95.3	(9.9)*	95.8	(6.3)*	96.3

Brackets indicate no reduction or increase

The test results (Table 3) also show that preoxidation using permanganate (Treatment A) had no significant effect on the removal of TOC. In contrast, permanganation increased color to unacceptable levels which suggest that the dosage (1.5 mgL^{-1}) is excessive. This is demonstrated by the high permanganate residual concentration (0.154 mgL^{-1}) which was measured in the GAS filtered water.

Preozonation (Treatment B) reduced color from 16 to 9 TCU, following coagulation/sand filtration. However, it had no significant effect on the reduction of TOC and the potential formation of THMs. These results are in agreement with Chang and Singer (1991) who studied the impact of ozonation on the removal of TOC and THM precursors. It is evident that color removal by ozone is not associated with the removal of precursors.

The THMFP test results confirm that preoxidation using either potassium permanganate or ozone had no significant effect on the removal of THM precursors. For example, the test results following sand and GAC filtration were similar to the control (non-oxidized) treatment. It was also observed that all treatments can remove turbidities to acceptable levels (1 NTU). It can be concluded that a preoxidation step is not require since there is no significant improvement in water quality.

The reduction of TOC and THMFP levels for the control and two treatments (A&B), expressed as a percentage of the average raw water samples (see Table 2), are summarized in Table 4. It is apparent that TOC can be used as a surrogate parameter of THM formation potential. The results also show that GAC filtration can effectively remove THM precursors and control the formation of THMs in Rosenort's drinking water. For example, the reduction in TOC for all treatments ranged from 87.0 to 93.0 mgL^{-1}. In comparison, the removal of THM precursors as measured by THMFP ranged from 95.8 to 96.3%. An average residual THMFP concentration of between 0.041 and 0.047 mgL^{-1} as $CHC1_3$ is observed in the finished water. The actual concentration of THMs which is likely to exist at the consumer's tap would be considerably lower.

The GAC media at the Rosenort WTP was replaced on October 27th, 1990. Three TOC measurements taken from raw and finished water samples after 2 months of operation indicate a reduction in TOC of less than 10%. The results of this experimental investigation suggest that the Lab GAC (Hydrodarco?3000) is more suitable to remove THM precursors than the plant GAC media. These observations indicate that the type of GAC media used is critical in controlling the formation of THMs in Rosenort's drinking water. Further laboratory testing of the Lab GAC column is, however, required to determine time to exhaustion, and TOC/THMFP breakthrough curves.

CONCLUSIONS

The conclusions of the bench scale tests are based on a limited amount of experimental data. However, the following general conclusions can be drawn based on the experimental results:

1. Coagulation using a cationic polymer (1.0 mgL^{-1}), followed by sand filtration and GAC (Hydroclarcs 3000) filtration reduced TOC levels by 10.9 and 88.2 percent, respectively.
2. Preoxidation using either potassium permanganate (1.5 mgL^{-1}) or ozone (2 mgL^{-1}) had no significant effect on the reduction of TOC and THM precursors as measured by TCMFP ($CHCl_3$).
3. GAC filtration can effectively remove THM precursors and control the formation of THMs in Rosenort's drinking water. Coagulation followed by sand/GAC filtration reduced THM precursors by approximately 96% resulting in an average THMFP of 0.047 mgL^{-1} as $CHCl_3$ in the treated water.
4. The type of GAC media used was shown to be critical in controlling the formation of TH's in Rosenort's drinking water.

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STANDARDS AND APPROVALS FOR MUNICIPAL WATER SUPPLY SYSTEMS IN ALBERTA

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1.0 INTRODUCTION

The development and implementation of an effective, and proactive, standards and approvals process, for public water supply systems, is perhaps one of the most challenging and important tasks that government regulatory agencies face today. The increased public concern regarding the "safeness" of public water supplies, combined with the seemingly ever increasing number treatment and quality issues associated with these systems, make it particularly important that standard and approvals procedures pertaining to drinking water supply systems be flexible and dynamic.

In Alberta, a formal approval process for the construction and operation of municipal water supply systems was established in 1971 with the proclamation of the Alberta Clean Water Act. The subsequent development and promulgation of associated Regulations in 1973 further defined and elaborated upon this approval procedure and also referenced specific standards applying to water supply systems. In the approximately 20 years that Alberta has had its Clean Water Act legislation, a standard setting and project approval procedure has evolved which is considered effective and also responsive to changing technologies and issues.

This paper presents an overview of Alberta's Clean Water Act legislation as it pertains to water supply projects and also outlines some of the specific technical standards that are applied to the various components of a water supply system. The general steps involved in the review and approval of a water supply project, from the initial feasibility assessments through to facility commissioning, are also outlined. New environmental legislation is currently being developed in Alberta and its potential implications to the review and approval of water supply systems is presented as is information on contemplated future changes to technical standards and the overall project review process.

2.0 LEGISLATED REQUIREMENTS

Alberta's Clean Water Act and associated regulations outline a number of specific regulatory procedures and requirements pertaining to municipal water supply systems. These include:

i) the requirement to obtain a Permit to Construct from Alberta Environment prior to the construction of any municipal water supply works;

ii) the requirement to design water supply systems in accordance with standards published by Alberta Environment;

iii) the requirement to obtain a Licence to Operate from Alberta Environment before commencing the operation of a water supply system; and

iv) the requirement that water supply systems provide water meeting the health-related quality limits in the Guidelines for Canadian Drinking Water Quality as published by Health and Welfare Canada.

These requirements provide a strong general framework for setting and implementing standards for water supply systems.

Alberta is currently in the process of developing new comprehensive environmental legislation that involves the consolidation of nine environmental acts including the Clean Water Act. The proposed title of this new Act is the: "Alberta Environmental Protection and Enhancement Act". The Act is tentatively scheduled for proclamation in the spring of 1992. This proposed new legislation will have a separate part that deals specifically with potable water and separate regulations will be developed that pertain only to drinking water. Under the current Clean Water Act legislation municipal facilities, i.e. water supply systems, storm water management works and sewage treatment disposal facilities are treated somewhat collectively. It is considered a desirable and positive step to separate drinking water legislation from wastewater management/control legislation.

The proposed Alberta Environmental Protection and Enhancement Act and associated regulations will incorporate most of the requirements and procedures of the existing Clean Water Act Legislation with some noticeable exceptions, namely:

i) the Permit to Construct and Licence to Operate approval requirements will be replaced with a single environmental approval covering both construction and operation;

ii) all project applications and/or approvals issued will be subject to a public notification/review process and an appeal process involving an Environmental Appeal Board; and

iii) the development and adoption of standards/guidelines/objectives will involve a public input/review process.

It is anticipated that the public review and appeal processes will result in comments and input regarding controversial or contentious water supply practices such as chlorine disinfection and fluoridation.

3.0 NUMBER AND TYPES OF REGULATED MUNICIPAL WATER SUPPLY SYSTEMS IN ALBERTA

3.1 Types of Water Supply Systems Regulated

The "inpractice" definition of a municipal water supply systems falling under the Clean Water Act legislation is:

> "Any communal municipal water system providing potable water to either a designated municipality (Hamlet, Metis Settlement, Village, Summer Village, Town, City), or any rural municipal development of two lots or more in which subdivision of the parcel has occurred for the purpose of establishing separate ownership of each lot (Alberta Environment, 1988)."

Some private developments on single deeded parcels of land and some water supply systems serving industrial operations are also permitted and licenced depending on the water supply source used and the type of water treatment provided. The Clean Water Act legislations applies to approximately 700 water supply systems throughout the Province of Alberta. The types of water supply facilities subject to regulatory requirements are not expected to change under the proposed new Alberta Environmental Protection and Enhancement Act.

3.2 Water Supply Sources and General Quality Issues

The Province of Alberta is covered by an extensive network of small and large rivers and numerous small to medium sized lakes many of which are used as water supply sources. Glacial deposits underlain by sedimentary rocks cover most of the Province and contain numerous aquifers that are also used as water supplies. Approximately two-thirds of the municipalities utilize surface water as a supply source and these surface water supply systems serve over 1.8 million people. In general, groundwater is used as a water supply source in smaller municipalities and non-municipal developments and serves approximately 300,000 people. The remainder of the 300,000 people in the Province are served by private water supply systems such as wells, cisterns receiving trucked water, or dugouts.

The major drinking water quality issues and challenges associates with surface water supplies in Alberta are turbidity removal, taste and odour problems, colour removal and trihalomethane formation. With respect to groundwater supplies, iron and manganese control and high total dissolved solids are the two most frequently encountered water supply quality issues.

3.3 Number and Types of Municipally Owned Water Supply Systems

In 1989 there were 271 municipally owned water supply systems in Alberta serving Cities, Towns, Villages, Summer Villages and Hamlets. Another 33 municipalities in the Province were served by regional water supply systems or receive water from another municipality. In addition, there were in excess of 400 non-municipally owned water systems serving developments such as schools, metis settlements, mobile home parks, provincial parks, water co-ops, commercial/industrial establishments, etc. Table 1 summarizes the number and different types of municipally owned water treatment systems in the Province.

Table 1. Type of municipally owned water treatment systems in Alberta.

WATER SUPPLY SOURCE	TYPE OF WATER TREATMENT SYSTEMS[1]	NO. OF SYSTEMS	POPULATION SERVED (approx.)
Groundwater	No treatment	23	10,000
	Chlorination only	47	41,000
	Chlorination and iron and/or manganese control	37	29,000
	Treatment that includes aeration	4	2,000
	Treatment that includes filtration and/or softening	10	23,000
	Totals	121	105,000
Surface Water	Filtration and chlorination	43	44,000
	Slow sand filtration and disinfection	8	4,000
	Coagulation, flocculation and/or sedimentation and filtration and chlorination	76	674,000
	Treatment that includes softening	12	965,000
	Treatment that includes carbon filtration	11	5,000
	Totals	150	1,692,000

Note: 1. 56 of these municipal systems fluoridate
2. An additional 33 municipalities (approximate total population 135,000) are served by regional systems or receive water from another municipality.

4.0 STANDARDS AND GUIDELINES

A major objective of Alberta Environment is to ensure that drinking water supplies and treatment systems provide a high level of public health protection while being able to meet the water supply needs of the community. In support of this objective the following three requirements have been established for all municipal water supplies in Alberta:

1) The water delivered to consumers must meet with the health related quality standards as outlined in the Health and Welfare Canada Guidelines for Canadian Drinking Water Quality;

2) The water treatment system must provide a basic level of protection against all possible sources and types of raw and treated water contamination; and,

3) Sufficient water must be available to meet the needs of the consumers, which may include fire protection.

With respect to drinking water quality, the Guidelines for Canadian Drinking Water Quality, published by Health and Welfare, Canada, have been adopted in Alberta, through the Clean Water Act legislation, as legally enforceable standards. In this regard, Section 23 of the Clean Water (Municipal Plants) Regulation states that:

> "The physical, microbiological, chemical and radiological characteristics of the water in a municipal water supply shall be maintained to meet the health-related concentration limits as contained in the Department of National Health and Welfare (Canada) publication entitled "Guidelines for Canadian Drinking Water Quality 1978."

An amendment to change the wording of this statement to refer the latest edition of the Guidelines will be incorporated in the new drinking water regulation that will be part of the proposed new Alberta Environmental Protection and Enhancement Act.

Reference to the aesthetic objectives (AOs) in the Guidelines were specifically excluded from the regulation as many municipal drinking waters in the province do not meet the AOs for parameters such as chloride, colour, iron, manganese, pH, temperature and total dissolved solids and it is considered impractical and unnecessary to try to meet these AOs in all situations.

To help ensure an adequate and safe drinking water supply that meets Canadian Drinking Water Quality, Alberta Environment has developed and published a document entitled "Standards and Guidelines for Municipal Water Supply, Wastewater and Storm Drainage Facilities (Alberta Environment, 1988)." Under the Clean Water Act, a municipal plant must be designed to meet any relevant standards in this document and/or any other standards established by Alberta Environment. Although the Standards and Guidelines document includes specific design criteria and standards that may have to be followed in the design of a water supply system, it is not intended to be a "design handbook" per se. The intent of these standards and guidelines is to focus on specific aspects of water supply that are considered critical in terms of protecting public health and meeting the water supply needs of a community. Specific criteria and guidelines are intended to assist both the regulator and the designer in achieving the objectives of public health protection and reliable water supply.

To the regulator, they are a means of specifying these requirements that are considered necessary for the treatment and delivery of potable water of acceptable quality; and to the designer, they provide useful guidance as to what the regulator expects both in terms of overall system design and in terms of criteria or specifications for various water supply/treatment processes and works.

A brief summary of the approaches and some specific design standards that are considered critical in the approval process is presented in the next few sections.

4.1 Design Capacity

Various components of water supply and treatment works should have a design life that is compatible with the function of the component. For instance, the treatment plant should be designed for a minimum period of 10 years with provision for expansion to handle a 20 to 25 year design flow. Intake structures, which have a high base construction costs, should be designed for at least a 20 to 25 year design flow. Treated water storage facilities should be phased to avoid operational problems such as increased chlorine demand or oversized pumps which are associated with excess storage. Establishing the local water consumption rates and accurately forecasting the population growth are key elements in determining the projected demand for different design horizons.

Regardless of design life, the raw water supply and the water treatment plants must be designed to meet the projected maximum daily demand for the design period.

4.2 Raw Water Supply

The key to selection of the raw water source is reliability and treatability. The raw water source should have sufficient capacity to supply water throughout the design period. The selected source must also be of good enough quality so that it can be economically treated to produce a finished water which complies with the Guidelines for with Canadian Drinking Water Quality.

A minimum of two pumps must be provided for raw water pumping, one being used as a standby. Pump capacities shall be such that with the largest pump out of service, the remaining pumps are able to supply the treatment plant demand.

A greater emphasis is being placed on the selection and development of good quality and reliable water supplies in an effort to reduce the reliance on treatment processes and operation to achieve quality requirements. This is often resulting in greater consideration of regional water supply options.

4.3 Water Treatment

Drinking water quality in Alberta is controlled by specifying both water quality standards and minimum treatment standards. The general minimum treatment requirement for all surface water supplies and all shallow well (i.e. less than 15 m) water supplies in Alberta is chemically assisted filtration plus disinfection. These requirements are based on the prevention of water-borne microbial diseases resulting from possible "assaults" on the water supply system.

Although these are the minimum treatment standards, many surface water supplies in the province are required to provide full physical and chemical treatment (i.e. chemical coagulation and flocculation, sedimentation, filtration and disinfection). Deep well supplies (i.e. greater than 15 m) require no treatment, however chlorination is recommended to protect against microbial contamination during distribution. Under the proposed new Alberta Environmental Protection and Enhancement Act all municipal water supplies witl distribution systems regardless of the water source will be required to disinfect to produce drinking water quality during distribution.

As the drinking water treatment system represents the "last barrier" against possible water-borne diseased, it is necessary that these systems be able to treat raw water of the "worst possible" quality, and this requirement is considered when establishing treatment system design standards.

4.3.1 Coagulation, Flocculation and Sedimentation

The proper pretreatment of water prior to filtration is considered essential to optimize colour and particle removal. Proper design of the rapid mix and coagulation process is critical for effective overall design of treatment system performance. Coagulation by inorganic salts occurs either by 'charge neutralization' or by 'sweep coagulation'. Charge neutralization reactions are extremely fast and it is imperative that the coagulant be dispersed in the raw water within a fraction of a second. This mode of coagulation at low dosages of chemicals produces small destabilized pinpoint floc which are ideal for direct filtration. For sweep coagulation, the coagulant is added to water in concentrations sufficiently high to cause precipitation of a metal hydroxide. Hydroxide formation occurs in the 1-7 second range and thus the short dispersion times and high intensities of mixing are not as crucial for sweep coagulation as for charge neutralization.

Coagulation by organic polymers occurs by charge neutralization. However, since the competing reactions for charge neutralization and sweep coagulation do not occur simultaneously, high intensities of mixing are not imperative when organic polymer is use as the coagulant. When organic polymers are used as primary coagulants they are added at the rapid mix unit and when used as a coagulant-aid they are added 1-3 minutes after the addition of the inorganic coagulant, very often at the flocculation unit.

No specific standards are currently applied to rapid mixers, however, more attention is now being given to the proposed design of this unit process in project reviews. Mechanical rather than static mix units are being strongly recommended based on past experiences.

Flocculation is required after rapid mix and coagulation for the aggregation of the destabilized collordal suspension. Flocculation times (t) vary depending on the velocity gradients (G), water temperature and whether or not sedimentation is provided. While a specific Gt value for flocculators is not specified, design Gt values outside the range 2×10^4 and 2×10^5 generally must be justified.

Sedimentation is designed to produce water which can be economically filtered by allowing for maximum filtration rates. The design criteria applied is that the tank must be designed with a retention time of at least one hour at design flow (much larger if colour is a problem) and with a depth of at least 3 m. No surface overflow rate is specified as this design parameter varies depending on the concentration of suspended solids, water temperature and the type of flocculation material. Lower rise rates are recommended for treatment of cold waters (less than 10°C); higher design rise rates are considered acceptable in conjunction with tube settlers or inclined plate settlers. For small plants with capacities less than 1000 m^3/d, the maximum rise rate allowed is 1.0 m/h.

4.3.2 Filtration

Filtration of all surface water supplies in Alberta is a minimum treatment requirement. Pressure filters are no longer permitted for municipal systems due to their inability to provide effective turbidity removal.

4.3.2.1 Rapid Gravity Filters

The following requirements are applied to rapid gravity filters:

- A minimum of two filters must be provided each capable of independent operation and backwash.
- Pre-treatment of the water with chemicals is required as a minimum i.e. on-line or direct filtration.
- An acceptable hydraulic loading rate depending on the quality of raw water and the extent of pre-treatment. (The maximum allowable filter rates are considered best determined by a pilot study using the proposed source water. High rate filtration systems exceeding 12 m/h are usually not allowed in Alberta.)
- Dual-media filters should meet the following general design criteria:

	Range	Typical
Anthracite:		
Depth, mm	300-600	450
Effective size, mm	0.8-2.0	1.2
Uniformity coefficient	1.3-1.8	1.6
Sand:		
Depth, mm	150-300	300
Effective size, mm	0.4-0.8	0.55
Uniformity coefficient	1.2-1.6	1.5

- Filter backwashing using potable water is required. (The backwash system should be capable of providing a minimum bed expansion of 50%. Air scour or surface auxiliary wash systems are not required but are generally installed to enhance the filter backwash system.)

- A filter to waste provision following the backwash cycle is required. After the filter is backwashed, a brief period of about 10 to 30 minutes is required to flush backwash remnants from the filter and allow filter ripening. This water must be wasted or recycled back to the pre-treatment works. It is recommended that filter to waste periods be based on actual filter turbidity measurements.

4.3.2.2 Slow Sand Filters

Because of its simplicity in design and operation and its potential to remove bacteria and cysts consistently from drinking water, slow sand filtration is receiving increased attention in Alberta. There are six standard slow sand filtration plants currently in service in Alberta and five of these are performing well. A further three slow sand filtration plants are currently being planned.

The main drawbacks with slow sand filters are their size and the fact that quick clogging makes them inappropriate for the treatment of high turbidity water or water with excessive growths of algae. The problem of high turbidity may be mitigated by providing a stilling basin before the filters. Algae growth is most difficult to control and may require pre-treatment, shading, chemical treatment or manual removal.

Alberta Environment has not yet established specific design criteria for slow sand filters but in general uses the following criteria when evaluating proposed slow sand filtration projects.

- Design criteria for the filters:

  Filtration rate - 0.04 to 0.42 m/h
  Specification of sand
     Effective size - 0.15 to 0.40 mm
     Uniformity coefficient - < 3
  Depth of filter bed
     Initial - 0.9 to 1.2 m
     Final - 0.6 m
  Depth of underdrains - 0.3 to 0.5 m
  Loss of head
     Initial - 0.6 m
     Final - 1.2 m

- Mode of Operation

  It is considered important that slow sand filters be designed for continuous operation. Intermittent operation is not permitted because it has been shown that in some cases an unacceptable breakthrough of bacteria can occur after the filters recommence operation.

- Number of Units

  At least two slow sand filters are required to ensure uninterrupted supply and to facilitate maintenance.

- Filter to Waste

  Soon after scraping of the filter bed, the treated water may not be of adequate quality to put into the distribution system. Thus, a filter to waste provision is required.

- Clear Water Tank

  In the past, some slow sand filters in Alberta have been built that are directly connected to the distribution system which results in variable filter loading rates and a varying quality of treated water. To avoid loading of the filter in accordance with the demand, a clear water tank to store treated water is now a general requirement.

- Air Binding

  Air binding in slow sand filter can occur in two ways. First, if the treated water outlet is at the same elevation as the bottom of the filter bed, negative pressure will occur in the bed resulting in air bubbles. This can be avoided by locating treated water overflows weir slightly above the top of the sand bed and this is now a general design requirement. The weir also ensures that the filter operates independently of fluctuations in the level of the clearwater reservoir.

  Air binding also results from refilling the filter bed from the top. Thus, provision is now required that the filter bed be refilled from the bottom up through the underdrain system.

4.3.3 Iron and Manganese Control

A significant number of groundwater sources in Alberta have high iron and/or manganese concentrations. The aesthetic objectives for iron and manganese in drinking water are 0.3 and 0.05 mg/L respectively as per the Guidelines for Canadian Drinking Water Quality (4th Edition). Groundwater sources containing higher concentrations may be blended or treated to achieve these aesthetic objectives. Some of the methods used to control iron and manganese in Alberta are listed below.

- Manganese Greensand Filters

  The most common form of iron and manganese control is by manganese greensand filtration with continuous regeneration of the media by potassium permanganate. No specific design standards for manganese greensand filters have been developed but the following basic design criteria for the filters are used when reviewing proposed filter designs:

  Depth of filter bed
    Manganese greensand - 375 to 600 mm
    Anthracite - 375 to 450 mm
  Filtration rate - 5.9 to 14.8 m/h
  Backwash - sufficient rate to produce 40% bed expansion
  Loss of head - maximum drop 1.2 m

Other proprietary material, similar to manganese greensand, such as birm and pyrolusite are also approved for use in Alberta to control iron and manganese in the water.

- Conventional Treatment Process

  Where iron and manganese are present in high concentrations or are complexed with organic compounds, removal by manganese greensand filters may be ineffective. In such cases conventional treatment processes are considered the only effective way to control iron and manganese. The processes may include aeration, chemical oxidation, sedimentation and filtration. Manganese cannot be oxidized as easily as iron and may require pH adjustment to about 8.5 for effective removal.

- Sequestering

  To maintain iron and manganese in soluble form and prevent them from coming out of solution, polyphosphate or silicate compounds may be added to drinking water to sequester these metals. However, most sequestering agents often require a narrow pH range for maximum effectiveness and may require a pilot testing to verify the suitability of the chemical. Sequestering has been experimented with extensively in Alberta with varying success. In general, Alberta Environment encourages bench or pilot-scale testing to determine the most effective method of iron and/or manganese control.

4.3.4 Disinfection

As stated earlier disinfection is required for all surface water supplies and shallow groundwater supplies. Disinfection of deeper well water is not required, however chlorination is recommended as a safeguard against potential contamination and/or growth of bacteria in the water distribution system.

Chlorine is the primary disinfectant in almost all the communities in Alberta. Chlorine must be applied at a point and at a dosage which will provide a minimum contact time of 20 minutes and produce a post contact free chlorine residual of 0.5 mg/L; the free chlorine level maintained in the distribution system must be 0.1 mg/L. When combined chlorine is used as the disinfectant, the treated water must have a combined chlorine residual of 1.8 mg/L after 20 minutes of contact time; the combined chlorine levels in the distribution system must be greater than 0.5 mg/L.

Chlorine dioxide may be used for either taste and odour control or as a pre-disinfectant. Total residual oxidants (including chlorine dioxide and chlorite, but excluding chlorate) shall not exceed 0.30 mg/L during normal operation or 0.50 mg/L during periods of extreme variations in the raw water supply. Because of possible health effects of chlorine dioxide and its by-products the use of chlorine dioxide is not being encouraged until health related chlorine dioxide residual and by-product standards are developed.

Ozone may be used as an oxidant for removal of taste and odour, or may be applied as a primary disinfectant. Ozone is not allowed to be used as a disinfectant in the absence of chlorine.

UV irradiation may be used as a primary disinfectant like ozone and chlorine dioxide, however, it may not be used as a disinfectant without the application of chlorine if the water is distributed through a piped system. UV radiation may be used as the sole method of water disifection at water points or in buildings.

Alberta has not yet considered the adoption of Ct (disinfection residual times contact time) criteria for water distribution. In some cases, however, consultants are encouraged to evaluate their proposed filtration/disinfection designs against the United States Environmental Protection Agency's (USEPA) requirements.

4.3.5 Fluoridation

Alberta has Fluoridation Regulations which govern the practice of water fluoridation. Before a public drinking water can be fluoridated, a plebiscite on the issue must be held and more than 50% of the votes must be in favor of fluoridation. The fluoridation regulation is quite detailed in terms of the type of equipment which should be used, the operation and monitoring of the fluoridation system, and the sampling and safety precautions associated with operating a fluoridation system. All of these requirements are also stipulated in the Alberta Environment's Standards and Guidelines.

Some of which are;

a. A fluoridation system shall include a "day tank" with a capacity to hold 24 hour supply of fluoride compound and the feeding system shall include an anti-syphon valves on the discharge side.

b. Adequate control and accurately calibrated feeding equipment shall be maintained at all times to provide a fluoride ion concentration in treated water of 1.0 mg/L. A monthly average variation from 1.0 mg/L of ± 0.1 mg/L and daily variation of ± 0.2 mg/L is acceptable.

c. Accurate daily recording of the following shall be carried out and submitted monthly to the Director of Standards and Approvals:

    (i) the daily reading of the water meter which controls the fluoridation equipment or that which determines the amount of water to which the fluoride is added;

    (ii) the daily volume of water fluoridated;

    (iii) the daily weight of fluoride compound in the feeder;

    (iv) the daily weight of fluoride compound in stock;

    (v) the daily weight of the fluoride compound fed to the water; and,

    (vi) the fluoride content of the raw and fluoridated water determined by laboratory analysis, with the frequency of measurement as follows:

    - treated water being analyzed daily, and
    - raw water being analyzed at least once a week.

The current fluoridation requirements in terms of the allowable daily and monthly variations are considered too restrictive and difficult to achieve from engineering standpoint. Under the proposed new Alberta Environmental Protection and Enhancement Act legislation, there will be no fluoridation regulation and the technical requirements pertaining to water fluoridation will be covered in standards. It is intended to change the standards for fluoridation to ensure that they are technically achievable.

4.4 Treated Water Storage

As water treatment plants are generally designed to satisfy only the maximum daily demand, treated water storage is provided in order to:

i) meet peak demands that exceed the capacity of the water treatment plant;

ii) ensure the continuance of water supply should there be any breakdown at the source or at the treatment plant.

Alberta Enivornment recommends using the following empirical formula in calculating the treated water storage requirements:

S = A + B + C

where S = Total storage requirement
A = Fire storage
B = Equalization storage (approximately 25% of projected maximum day demand)
C = Emergency storage (minimum of 15% of projected average day demand)

Fire storage is not mandatory from a regulatory standpoint and, it is the responsibility of the municipality to determine its need. Further, in addition to the calculated storage, an allowance must also be made for the operation of the water treatment facility, i.e. filter backwash.

4.5 Watermain and Transmission Main

Pipe Material

It is a requirement in Alberta that where a Canadian Standards Association (CSA) standard for a particular pipe material and size exists, any such pipe used for potable water systems shall be 'certified' for potable water use by CSA, and that the pipe shall be marked accordingly. This certification process ensures that the pipes are manufactured from a resin conforming to the applicable CSA Standards.

Pipe Sizing

Alberta Environment's standards stipulate that watermains designed to carry fire flows must be a minimum of 150 mm; and that for smaller distribution systems without a fire flow provision, watermains must be designed to handle a normal operating pressure between 350 kPa and 550 kPa under a condition of maximum hourly flow. In order to avoid damage to household piping, maximum system pressures should not exceed 700 kPa. Further, to provide adequate service during a fire, the minimum distribution pressure during peak demand flows must be 150 kPa.

Pipe Layout

Some design criteria for the pipe layout are as follows:

1. Water distribution systems should be designed to eliminate dead end sections; and blow off provision should be provided on all lines that are dead-ended.
2. Air release valves should be provided at all significant high points and drain valves should be placed at low points of large mains.
3. The cover for watermains should not be less than 2.5 m.
4. To preclude the entrance of contaminants into the water distribution system, the watermains should be laid with at least 3 m horizontal separation from any storm or sanitary sewer. Exception may be made to reduce the horizontal distances under unusual conditions.
5. Under normal conditions watermains should be designed to cross above sewers. Where it is necessary to cross below the sewer, a vertical separation of at least 0.5 m from watermain crown to sewer invert must be provided.

5.0 APPROVAL PROCESS

The existence of regulatory standards and guidelines for municipal water supply systems does not necessarily ensure that municipal facilities will be designed, built and operated to these standards. In Alberta, it is a statutory requirement, under the Clean Water Act, for the ultimate owner of a facility to obtain an approval from Alberta Environment before commencing construction or changing any municipal water supply works. It is also a requirement for municipalities to obtain written permission from Alberta Environment before adding any new chemicals to the water or before changing the treatment process. A major responsibility of Alberta Environment is to conduct project reviews to ensure that the required municipal waterworks are designed to meet standards prior to issuance of the required 'Permit to Construct'; and that chemicals proposed are approved for potable water use prior to issuance of the "Letter of Permission".

A municipal water project may be initiated by the communities as a result of community growth or it may be initiated by Alberta Environment to address concerns with the water treatment facility or unacceptable treated water quality which are monitored and reported to Alberta Environment by municipalities as per their licence requirements .

The Alberta Environment "Permit to Construct" review process usually starts at the very inception of a project. This is termed the facilities planning stage, and involves an inventory and assessment of existing works and identification of upgrading and/or expansion alternatives. In addition to verifying the engineering feasibility of conceptual alternatives, evaluations are undertaken to determine the relative cost-effectiveness of each alternative since projects are government funded.

It should be noted that Alberta Environment does not require the immediate implementation of a new water treatment or quality standard at a facility unless implementation is relatively straightforward or there is a health risk that must be addressed. Any relevant new drinking water treatment or quality standard is generally applied at the time of a major facility upgrading or expansion, and these requirements are outlined to the municipality at the facility planning stage. At the end of this project stage, the municipality or its agent may make an application to Alberta Environment for a Permit to Construct. A typical application generally includes the following:

a. an engineering report to establish the need for, and the feasibility of, the project;
b. detailed plans and specifications stamped and signed by a Professional Engineer registered in Alberta; and
c. detailed cost estimates.

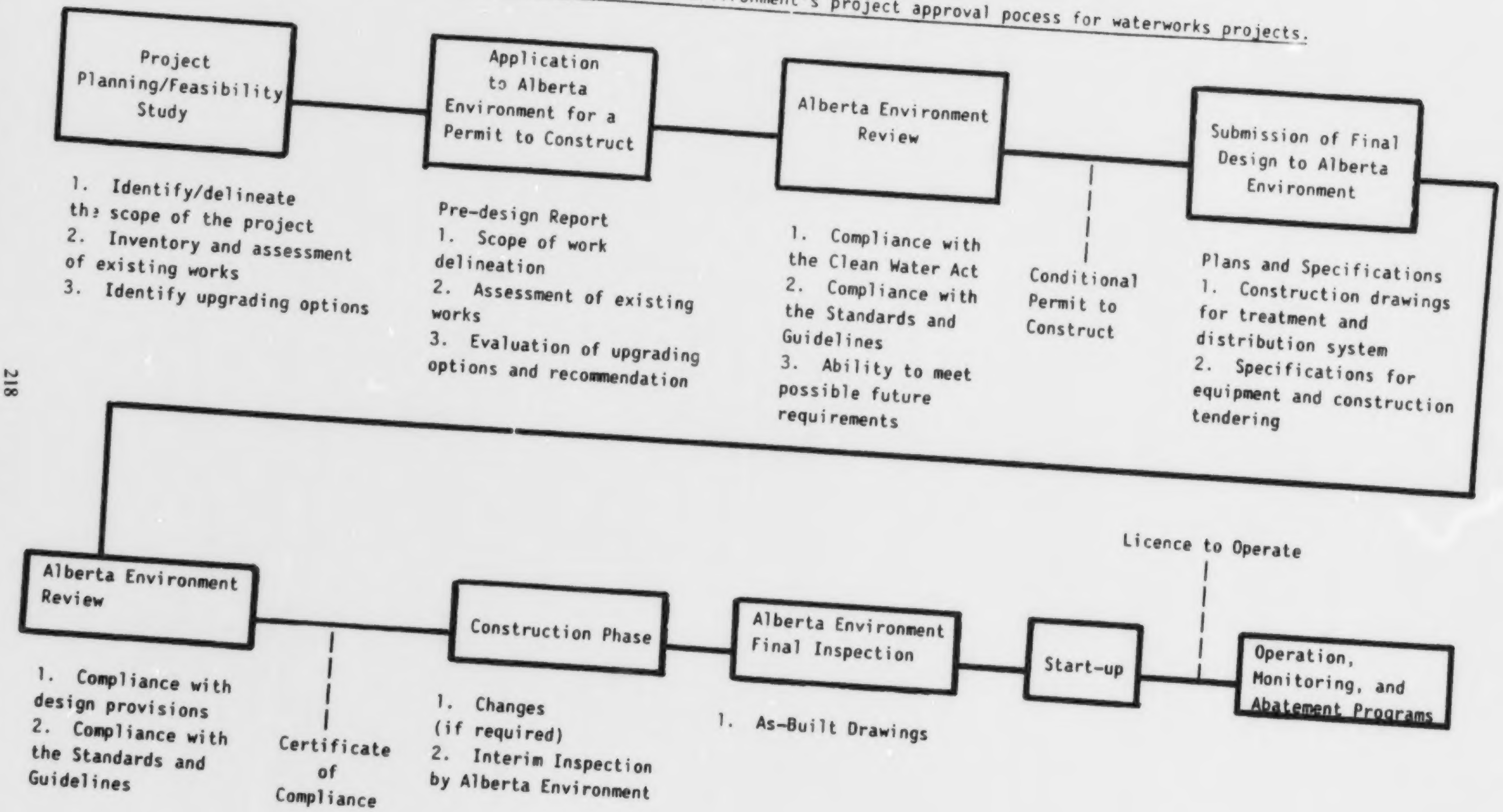
If the detailed plans and specifications cannot be submitted at this time, a conditional Permit to Construct may be issued with the final approval of the project prior to construction, contingent upon submission of detailed plans and specifications. The removal of this or other relevant conditions from the list of permit conditions is accomplished by the issuance of a Certificate of Compliance. The owner of the system is also advised, at this stage, that near completion of the project construction, and prior to commencement of operation, an application must be made to Alberta Environment for a Licence to Operate for the facility.

Figure 1 summarizes the steps generally involved in obtaining the necessary approvals from Alberta Environment for the construction of a water supply system. Examples of a typical "Permit to Construct" and a "Certificate of Compliance" are presented in Appendix A and B respectively.

The Alberta Environment "Letter of Permission" is issued whenever the treatment process is changed or any new chemicals are added to enhance treatment of water. The need for such improvements are evaluated through regular inspections of the municipal water facilities by Alberta Environment staff and by reviewing monthly reports that are submitted by the communities to Alberta Environment as per their Licence to Operate requirements. After identifying a concern with the operation of the facility, Alberta Environment will work with the communities to optimize the plant operation or if required, try out new chemicals to enhance the treatment process. All chemicals used in the municipal water systems in Alberta must meet National Sanitation Foundation (NSF) standards 60 and 61 for direct and indirect additives. Example of a typical Letter of Permission is presented in Appendix C.

The Permit to Construct and the Letter of Permission program offers Alberta Environment an opportunity to provide advice or guidance to municipalities, based on current water treatment trends. In particular, advice on possible future treatment and quality requirements that could possibly be considered when designing and constructing water supply works is provided. The program, therefore, indirectly assists in helping to minimize the impact, and to facilitate implementation of future changes to the Guidelines for Canadian Drinking Water Quality.

Fig. 1. Schematic flow diagram - Alberta Environment's project approval pocess for waterworks projects.


Project Planning/Feasibility Study
1. Identify/delineate the scope of the project
2. Inventory and assessment of existing works
3. Identify upgrading options
Application to Alberta Environment for a Permit to Construct
Pre-design Report
1. Scope of work delineation
2. Assessment of existing works
3. Evaluation of upgrading options and recommendation
Alberta Environment Review
1. Compliance with the Clean Water Act
2. Compliance with the Standards and Guidelines
3. Ability to meet possible future requirements
Conditional Permit to Construct
Submission of Final Design to Alberta Environment
Plans and Specifications
1. Construction drawings for treatment and distribution system
2. Specifications for equipment and construction tendering
Alberta Environment Review
1. Compliance with design provisions
2. Compliance with the Standards and Guidelines
Certificate of Compliance
Construction Phase
1. Changes (if required)
2. Interim Inspection by Alberta Environment
Alberta Environment Final Inspection
1. As-Built Drawings
Start-up
Licence to Operate
Operation, Monitoring, and Abatement Programs

6.0 SUMMARY

Through the development and application of water supply system design criteria and guidelines, Alberta Environment has been able to control and dictate certain aspects of water supply system design which it believes are critical to ensuring the provision of safe drinking water. For some components of water supply systems specific minimum criteria are established, e.g. minimum treatment requirements and minimum disinfectant contact times and residuals, and for other components, e.g. filtration, specific design criteria have not been established but certain generally accepted design criteria are used when evaluating a proposed project.

Alberta's Clean Water Act legislation has been found to provide an effective regulatory framework for applying and implementing general and specific requirements for municipal water supply systems. This legislation's Permit to Construct and Licence to Operate requirements gives Alberta Environment staff the opportunity to be involved in all aspects of the planning and implementation of a project. New environmental legislation is being introduced in Alberta but it is not expected that this new legislation will alter significantly the general approach and philosophy currently followed with respect to the review and approval of municipal water supply systems. The major change under the proposed new legislation will be the provision for public input into both the development of standards and the approval of water supply.

In conclusion, a strong; yet somewhat flexible, regulatory framework combined with appropriate design standards and guidelines are considered to provide Alberta Environment with the tools necessary to ensure that municipal water supply systems in the Province of Alberta meet good basic physical design requirements and are capable of providing safe drinking water under all foreseeable operating conditions.

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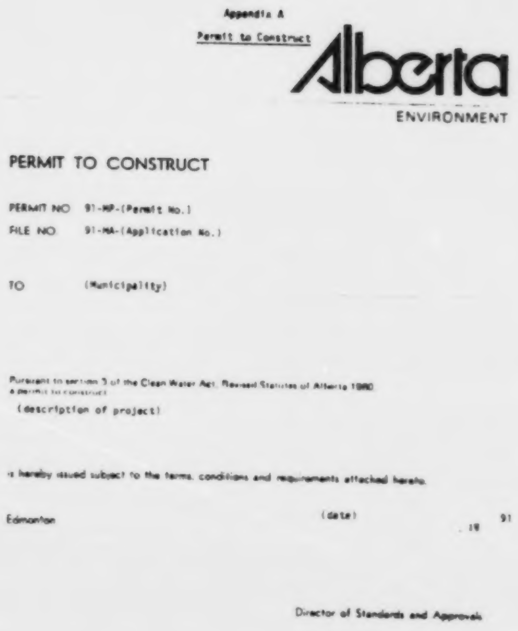


Appendix A

Permit to Construct

Alberta
ENVIRONMENT

PERMIT TO CONSTRUCT

PERMIT NO 91-MP-(Permit No.)

FILE NO 91-MA-(Application No.)

TO (Municipality)

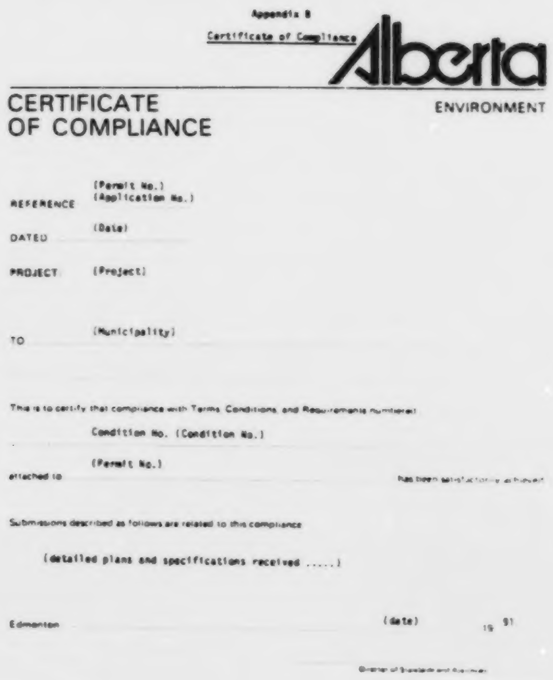
Pursuant to section 3 of the Clean Water Act, Revised Statutes of Alberta 1980 a permit to construct

(description of project)

is hereby issued subject to the terms, conditions and requirements attached hereto.

Edmonton (date) , 19 91

Director of Standards and Approvals



Appendix B

Certificate of Compliance

Alberta
ENVIRONMENT

CERTIFICATE OF COMPLIANCE

REFERENCE (Permit No.) (Application No.)

DATED (Date)

PROJECT (Project)

TO (Municipality)

This is to certify that compliance with Terms, Conditions, and Requirements numbered

Condition No. (Condition No.)

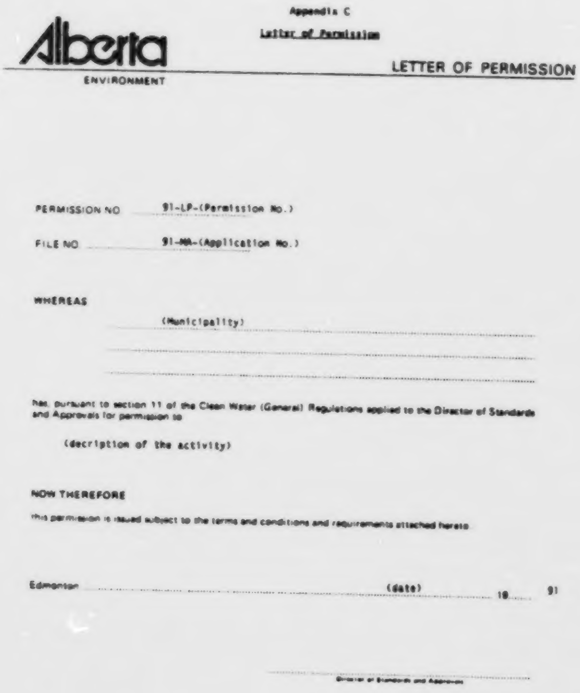
attached to (Permit No.) has been satisfactorily achieved

Submissions described as follows are related to this compliance

(detailed plans and specifications received .....)

Edmonton (date) 19 91

Director of Standards and Approvals



Appendix C

Letter of Permission

Alberta
ENVIRONMENT

LETTER OF PERMISSION

PERMISSION NO 91-LP-(Permission No.)

FILE NO 91-MA-(Application No.)

WHEREAS

(Municipality)

has, pursuant to section 11 of the Clean Water (General) Regulations applied to the Director of Standards and Approvals for permission to

(decription of the activity)

NOW THEREFORE

this permission is issued subject to the terms and conditions and requirements attached hereto

Edmonton (date) 19 91

Director of Standards and Approvals

REMEDIATION OF GASOLINE CONTAMINATED GROUND WATER

Edward A. Wolowich, M.Sc., P.Eng.
Wardrop Engineering Inc.

Brent K. Horning, B.Sc., P. Eng.
Wardrop Engineering Inc.

SUMMARY

A loss of approximately 15,000 litres of gasoline from underground storage tanks resulted in the contamination of an estimated 3,000 m^3 of soil and the formation of a liquid gasoline plume which occupied over 1,000 m^2 of an aquifer at Carberry, Manitoba. A remediation program was implemented in 1985 to recover the gasoline from the soils and aquifer. The objective of the remediation program was to first contain the gasoline plume and secondly to return the aquifer to a usable state.

Following the installation of several monitoring wells, and computer modelling of the contaminant plume, a plan of remediation was devised. A recovery system was designed and implemented, which comprises 2 water table depression and recovery wells, a hydrocarbon vapour extraction system and an air stripping tower.

Since implementation of the recovery systems, the subsurface liquid gasoline plume has been reduced to isolated pockets. However, following rain events, gasoline continues to leach from the contaminated soils and move downward to the water table, resulting in intermittent floating liquid plumes.

The reduction in the size of the initial liquid plume is chiefly attributed to the operation of the recovery systems in place. The vapour extraction system had removed approximately 4,000 litres of gasoline from the soils above the aquifer, preventing the gasoline from seeping into the aquifer and adding to the liquid plume. The ground water extraction system was then able to reduce the liquid plume significantly, through the removal of approximately 3,000 litres of gasoline from the aquifer.

The size fluctuation of the liquid gasoline plume is an indication that the volume of gasoline remaining in the subsurface is approaching non-recoverable or residual amounts. Continued use of the recovery systems is expected to remove the recoverable gasoline contamination from the subsurface within the next two years. This would include removal of the remaining free product or liquid gasoline floating on the water table, most of the gasoline adsorbed or suspended by the soils above the water table, and most of the gasoline dissolved in the ground water. Residual amounts, which can not practicably be removed, will remain and will continue to biodegrade with time.

INTRODUCTION

Site History

In 1985 the domestic well water of a house adjacent to a former service station became contaminated with gasoline. The well water was analyzed and found to contain 0.4 parts per million (ppm) of dissolved gasoline. Four underground gasoline storage tanks were still present at the former service station. These tanks were pressure tested and found to be leaking. Two of the tanks, which had not been used since 1980, contained several hundred litres of gasoline. The tanks were then removed.

Manitoba Environment installed nine 50 mm diameter monitoring wells in the vicinity, which revealed a gasoline plume contaminating the aquifer. As much as 1 m of gasoline was detected, floating on the water table in some of these wells, while others contained only dissolved product at concentrations in excess of 4 ppm. An additional nine monitoring wells were installed, in an attempt to determine the extent of the plume. A 760 mm diameter recovery well was then installed and periodically pumped, to remove the gasoline from the aquifer. However, due to a rather low well yield the program was largely unsuccessful.

From the monitoring well installations, it was estimated that 15,000 litres of gasoline had contaminated the soils and aquifer at the spill site. This gasoline contamination is present in three distinct phases (Figure 1): dissolved, floating, and suspended. As the gasoline released from the storage tanks moved downward through the soils above the aquifer (referred to as the vadose zone), some gasoline was adsorbed within the soil structure. This phase of the gasoline will remain suspended by the soils while the remaining, free gasoline moves downward to the water table. When the water table is encountered most of the gasoline not adsorbed by the soils will float on the surface of the ground water. This portion of the contaminant is referred to as the floating phase or free product phase. However, gasoline is slightly soluble in water and some gasoline will become dissolved in the ground water. This dissolved phase or soluble phase will travel with the ground water and can affect users of the ground water for a significant distance from the spill site. Often three separate recovery systems are needed to capture these three phases of the contaminant. Further, it is usually not possible to recover all of the gasoline from the subsurface, without actually removing the contaminated soils.

Hydrocarbon vapours emanating from the gasoline adsorbed by the soils migrated throughout the immediate area. Since domestic water in Carberry is supplied by private wells, the contamination threatened the quality of the domestic water supplies for a school, a senior citizen's home, a hospital and private residences.

Since January 1988, the remediation measures have been modified and supervised by Wardrop Engineering Inc. The cleanup system now comprises: 28 monitoring wells to track the contamination; two ground water table depression and liquid gasoline recovery wells to recover the floating plume; a vapour extraction system to recover the suspended or adsorbed gasoline, and an air stripping system for removal of the dissolved gasoline from the ground water.

Setting

The Town of Carberry is located in south-central Manitoba on the Assiniboine Aquifer (Figure 2). The Aquifer is an extensive, water bearing sand, deposited by the Assiniboine River as it flowed into Lake Agassiz during glacial melt. The fine to medium sands of the aquifer are usually exposed at the surface, with the ground water being unconfined. However, in the vicinity of Carberry, local flood plain and lacustrine deposits of silt and clay cover the aquifer to thicknesses of 0.5 to 4 m (Figure 3). The saturated thickness of the Assiniboine Aquifer reaches a maximum of over 30 m slightly northwest of Carberry, while the saturated thickness at the Town approaches 15 m.

Data collected throughout the aquifer region during 1964 and 1965 revealed that the ground water flow in the vicinity of Carberry was nearly due east (Pederson, 1970). Testing performed on two wells at a former Air Force base southwest of Carberry revealed well transmissivities of 256 and 290 m^2/day. The specific capacity of these wells is between 0.5 and 0.6 l/s. In contrast, the values for other regions of the Assiniboine delta aquifer range as high as 1400 m^2/day. Consequently, while aquifer parameters in the vicinity of Carberry show the delta sand to be a good and usable aquifer, other areas of the aquifer are capable of generating far higher ground water yields.

At the spill site the static water level is approximately 5 m below grade.

A number of relatively high volume ground water users were identified in the Town of Carberry. These include a Personal Care Home and Hospital complex east of the spill site, a senior's apartment block to the south, a senior high school southeast and, an elementary school east of the site (Figure 4). Intensive pumping at these sites could cause the migration of the gasoline plume toward them. Further, based on the 1964-65 data the Elementary School, Hospital and Personal Care Home are downstream of the gasoline plume. The contaminant plume, as it was in 1987, is shown on Figure 5.

COMPUTER MODELLING

An analytical model (Wilson and Miller, 1978) was used to predict gasoline concentrations over the study area, particularly to determine the gasoline concentrations at the downstream users (i.e., the hospital, personal care home and elementary school). The aquifer parameters used in the model were those derived from pumping tests of the former Air Force base wells. For simplicity the aquifer transmissivity was held constant, although in reality this is rarely the case. Under homogeneous aquifer conditions and a constant background flow rate the model predicted gasoline concentrations approaching 1 ppm at the hospital within one year.

Since it can safely be assumed that the underground storage tanks were leaking for at least one year prior to discovery, and the hospital water did not test positive for aromatic hydrocarbons, the model predictions did not seem valid. The difference was likely due to the heterogeneous nature of the aquifer, which contains silt and clay zones that retard the migration of the contaminant. Further, since the aquifer is unconfined to semi-confined, natural degradation of the contaminants would likely take place while migrating any great distance. Nevertheless, it was deemed appropriate to contain and recover the gasoline.

In order to devise the most appropriate containment and recovery techniques, the aquifer was modelled utilizing a finite difference routine (Prickett and Lonquist, 1971). The model simulated the apparent ground water flow conditions, as impacted by the surrounding users, and generated a piezometric (ground water) surface plot, as shown in Figure 6. The model was then used to predict the migration of the gasoline plume and determine the effect of recovery wells placed at various locations. The impact of a proposed Senior's apartment block on the contaminant plume was also studied. Through these computer simulations the required pumping rate and recovery well locations were determined.

RECOVERY SYSTEM DESIGN AND OPERATION

Free Product Recovery

The liquid gasoline or free product plume is recovered from the aquifer by depressing the surface of the water table and skimming off the liquid gasoline that collects in the resulting depression (Figure 7).

Two large diameter wells, the North Recovery Well (610 mm diameter) and the East Recovery Well (760 mm diameter) equipped with ground water table depression pumps were installed. Both wells, constructed of perforated steel cribbing, were drilled to approximate depths of 15 m.

Pumping these wells creates a "cone of depression", which is a lowering of the water table around the wells, thereby enhancing the flow of liquid gasoline from the surrounding contaminated area into the wells. Once a sufficient volume of liquid gasoline has accumulated in the wells, it is skimmed off the surface of the water with a product pump and stored in a temporary, mobile storage tank.

Vapour Extraction

Adsorbed gasoline is recovered from the soils by means of a vapour extraction system. The system lowers the pore pressure within the soils by suction and allows the adsorbed gasoline to vapourize or volatilize. The system draws these vapours from the soils, like a vacuum, and expels them to the atmosphere (Figure 8).

The West Recovery Well is equipped with a 2,000 l/s extraction blower. This well is also interconnected, through underground 50 mm diameter PVC piping, to several of the 50 mm diameter, monitoring wells as well as the North Recovery Well. This system allows the blower to withdraw gasoline vapours from the vadose zone (the unsaturated soil above the ground water table) over an extensive area, along the east side of the former service station (the former location of the underground storage tanks). The blower has been in continuous operation since installation in the spring of 1987. Figure 9 shows the gasoline concentrations in the exhaust of the extraction system through 1990.

In addition to the blower, the West Recovery Well is equipped with an air injector. This unit is used to pump air into the ground water within the well. The resulting turbulence frees some of the dissolved gasoline from the ground water. The resultant vapours are withdrawn from the well by the blower. The air injector also adds oxygen to the aquifer, which aids in the biodegradation of the dissolved gasoline. The air injector has been in use since the summer of 1988.

Air Stripping

A water treatment process referred to as air stripping was implemented at the spill site to remove dissolved gasoline from the contaminated ground water (Figure 10).

An air stripper tower was installed west of the former service station and was put into operation in June of 1990. The system is designed to remove the aromatic hydrocarbons in the ground water, which is being pumped from the North and East Recovery Wells. The water discharged from these wells is sprayed into the top of the 10 m vertical tower, filled with plastic packing. Air is injected upwards from the base of the tower which evaporates or volatilizes the gasoline (aromatic portion) that adheres to the plastic packing by surface tension. The gasoline vapours (aromatic hydrocarbons) are then forcibly ejected out of the top of the tower.

Once engaged, the air stripper tower was supplied with contaminated ground water at a rate of 4.1 l/s. The discharge from the tower was then split between a return well installed nearby (1.1 l/s) and a discharge line to a ditch north of Town (3.0 l/s). Injecting water into the return well, upstream (with respect to the ground water flow regime) of the gasoline plume, helps force the gasoline plume east to the recovery wells where it is pumped from the aquifer. These installations are shown on the Figure 11 Site Plan.

REMEDIATION RESULTS

Free Product Plume

The water table depression pump in the East Recovery Well has been in nearly continuous operation since 1987, while that in the North recovery well was installed in 1989. Through these recovery wells approximately 3,000 litres of liquid gasoline have been recovered from the aquifer to date.

The extent of the product plume was reduced from over 1,000 m² to small isolated pockets. However, the decrease was far from being linear with time. The free product plume was completely removed from time to time, and has reappeared following significant rain events. It is likely that following the removal of a significant portion of the free product plume, precipitation has leached additional product into the aquifer from the contaminated soils above. The vapour extraction system appears to be effective in removing much of the leachable gasoline from these soils, since the reappearance of a free product plume is becoming less frequent.

Adsorbed or Suspended Phase

A Gastechtor Model 1238 Hydrocarbon Surveyor, calibrated to hexane, was used to measure the hydrocarbon vapour concentrations in the monitoring and recovery wells as part of monthly site inspections. Vapour concentrations were found to be wide-ranging, from non-detectable (less than 2 ppm) to in excess of 100 percent of the lower explosive limit (% LEL) for hexane, which is the Gastechtor unit's upper measurement limit (100% LEL = 12,500 ppm).

A contour plot of the gasoline vapour plume in October, 1990, presented as Figure 12, shows that the vapour concentrations are highest beneath Main Street, to the east of the adjacent residence, with a second elevated area near the northeast corner of the former service station. Fortunately, these gasoline vapour concentrations drop off rapidly, and therefore pose minimal danger to the occupants of the surrounding buildings. The elevated concentrations found along the east wall of the former service station, however, are of concern. If the vapour extraction system is turned off, gasoline vapours along the east wall can build up to 100% LEL. The continued use of the vapour extraction system is controlling this situation.

The effect of temperature on the vapour concentrations is quite noticeable. During the winter months, vapour concentrations are significantly lower than those recorded during the summer months (Figure 9). Monthly monitoring of the hydrocarbon vapour concentrations in the blower exhaust have shown values ranging from 5 to 150 ppm. These concentrations are dependent on the outside air temperature; the colder the temperature, the less volatile the gasoline and, therefore, the lower the vapour concentrations emitted by the vapour extraction system.

Using the hydrocarbon vapour concentrations recorded and the ambient air temperature, the gasoline vapours expelled by the blower were converted to equivalent liquid volumes. These calculations show that since implementation the equivalent of 4,000 litres of liquid gasoline have been extracted from the vadose zone.

Vapour extraction rates are now decreasing. It is believed that the contamination is approaching residual amounts. Recovery of the gasoline, particularly from the soils, will become increasingly difficult. Laboratory testing has shown that the residual retention for packed sand is between 12% and 60% of the pore volume (Hoag and Marley, 1986). Therefore, it may not be practicable to recover these residual amounts (barring excavation of the contaminated soils). Natural degradation of the gasoline, both biological and physical, will continue for quite some time.

Dissolved Product Phase

The effectiveness of the air stripper system has been periodically determined through laboratory analyses of the supply and discharge water. Based on initial results, it was found that the ground water being injected into the tower was relatively uncontaminated prior to treatment. The water table depression pumps supplying the air stripper were then raised, in order to withdraw water closer to the water/gasoline interface. Following this change both laboratory analyses and on-site inspections (noticeable hydrocarbon odors in the vicinity of the air stripper tower), showed the system to be operating effectively. Samples were collected from the recovery well discharge line to the air stripper and from both a surge tank located downstream of the air stripper and the ditch discharge outlet. These water samples were analyzed for aromatic hydrocarbons (benzene, toluene, ethylbenzene and xylenes). Further samples were analyzed for dissolved gasoline content. These analyses showed the air stripper to be reducing 0.24 ppm dissolved gasoline in the ground water to non-detectable levels (below 0.5 ppb).

Water samples were also collected from domestic water supplies around the spill site in November 1990. These water samples were collected from the former service station, the adjacent residence, a Senior's apartment block, the elementary school and the ditch discharge. These samples, along with a field blank of commercially available distilled water, were analyzed for aromatic hydrocarbons. The results of this analysis (Table 1) revealed that through the containment and recovery program, the gasoline spill resulted in only minor effects on the water quality at those sources. The contaminant concentrations recorded were well below the Canadian Drinking Water Quality guidelines for maximum acceptable concentrations. This suggests that there is no health danger to persons who continue to consume the water from these sources.

CONCLUSIONS

1. Approximately 7,000 litres of gasoline have been recovered from the aquifer since implementation of recovery measures; 4,000 litres through vapour extraction and 3,000 litres through free product recovery.

2. The amount of gasoline remaining in the soils is approaching residual amounts. Most of the remaining gasoline is adsorbed by the soils and will only be recovered during periods of high precipitation, when the gasoline is leached from the subsurface soils, or during periods of warm temperatures, when the gasoline volatilizes and is expelled by the vapour extraction system.

3. The existing free product recovery and vapour extraction systems are operating efficiently and are effectively reducing the amount of free and suspended gasoline remaining in the subsurface.

4. Air stripping is effective in reducing dissolved gasoline concentrations of up to 0.24 ppm to non-detectable levels.

5. Natural biodegradation of the remaining residual plume will continue, following recovery of the non-residual portion.

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ACKNOWLEDGMENTS

The authors wish to acknowledge Federated Insurance Company of Canada, particularly Mr. J. Casey and Mr. B. Bohunicky, for whom this work was undertaken.

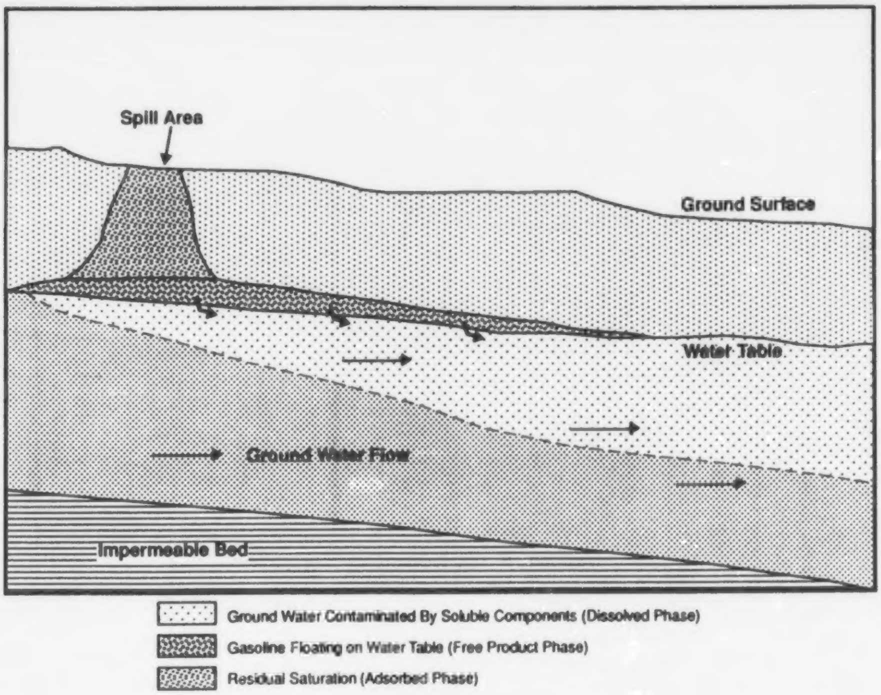

Spill Area
Ground Surface
Water Table
Ground Water Flow
Impermeable Bed
Ground Water Contaminated By Soluble Components (Dissolved Phase)
Gasoline Floating on Water Table (Free Product Phase)
Residual Saturation (Adsorbed Phase)

Figure 1
Gasoline Contamination of Soils and Ground Water

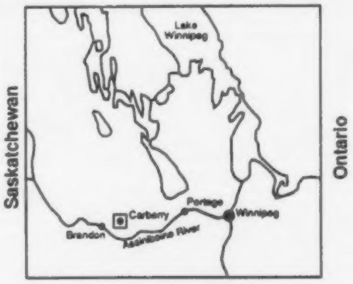


Figure 2
Study Area

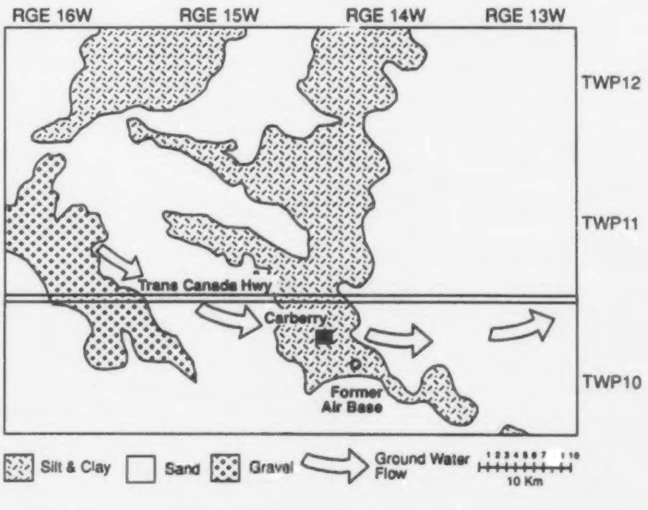


Figure 3
Surficial Geology

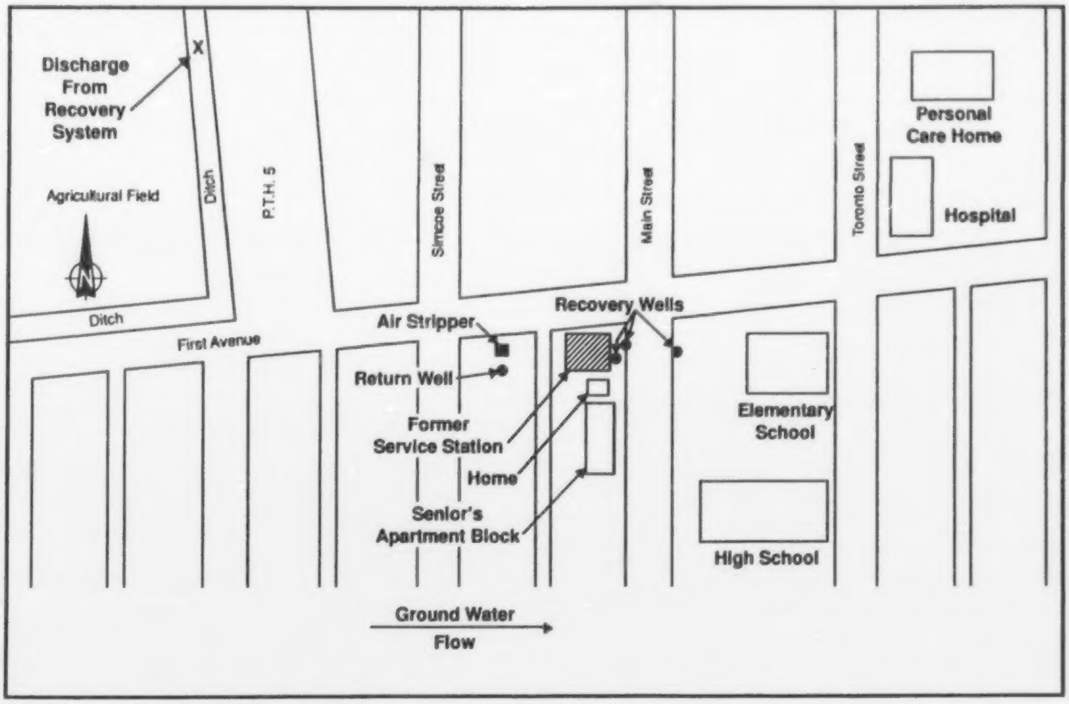


Figure 4
Site Plan Showing Adjacent Land Use

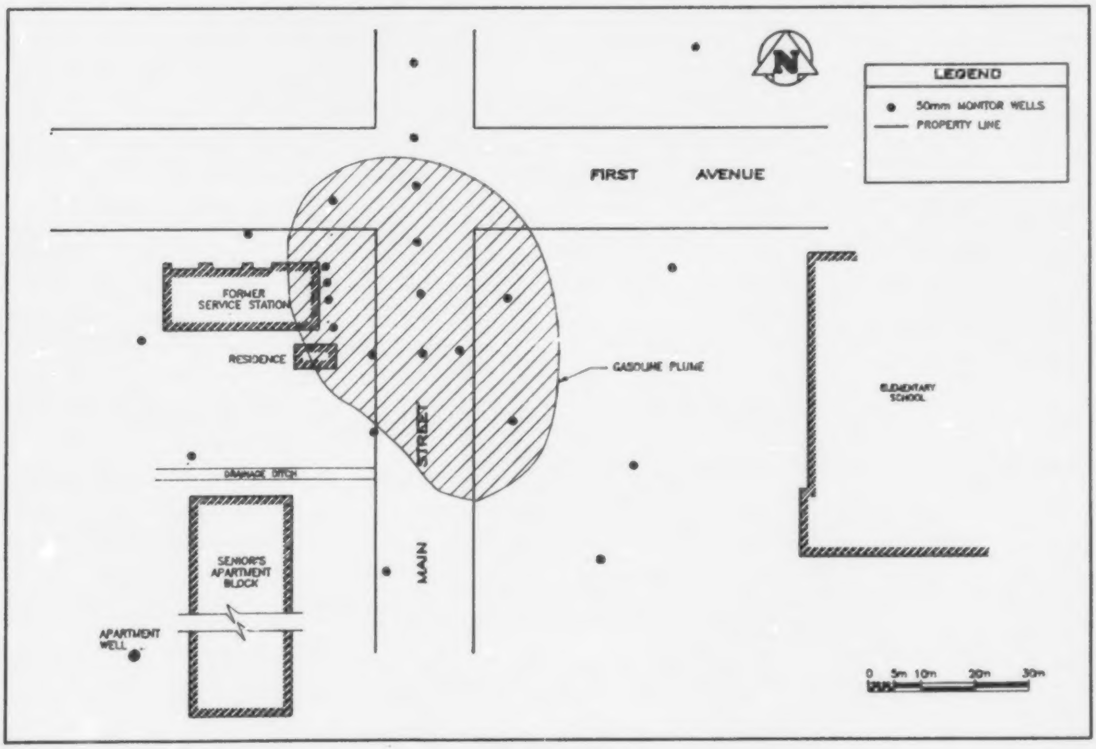


Figure 5
Free Product Plume, 1987

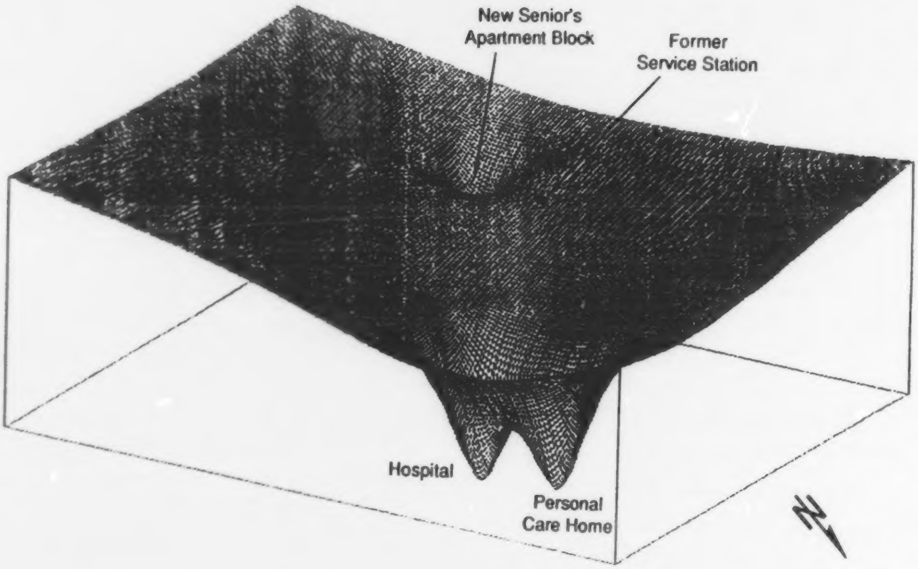


Figure 6
Computer Simulation of Piezometric Surface

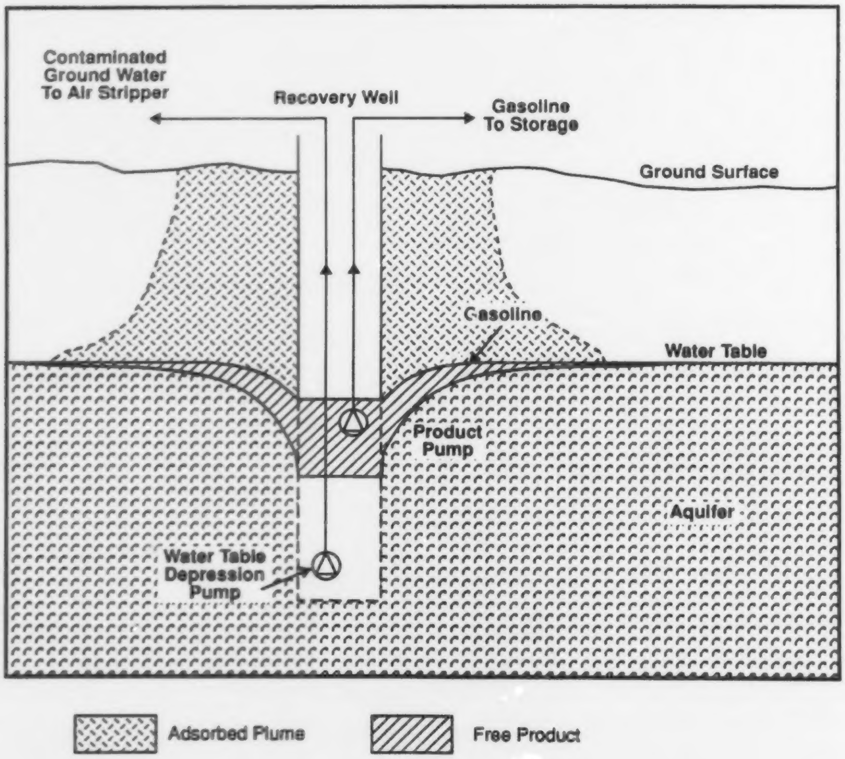


Figure 7
Free Product Recovery

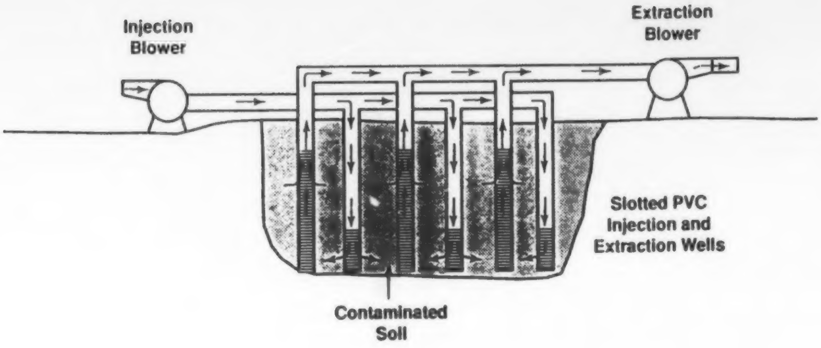
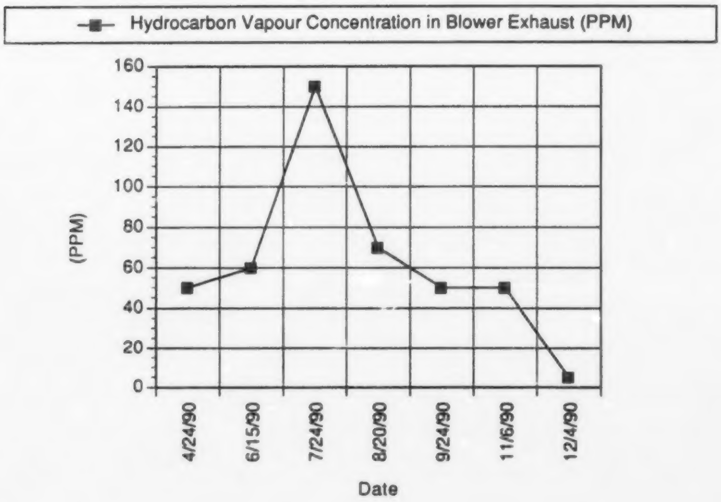


Figure 8
Typical Vapour Extraction System



Date	(PPM)
April 24/90	50
June 15/90	60
July 24/90	150
August 20/90	70
September 24/90	50
November 6/90	50
December 4/90	5

Figure 9
Vapour Extraction System Performance

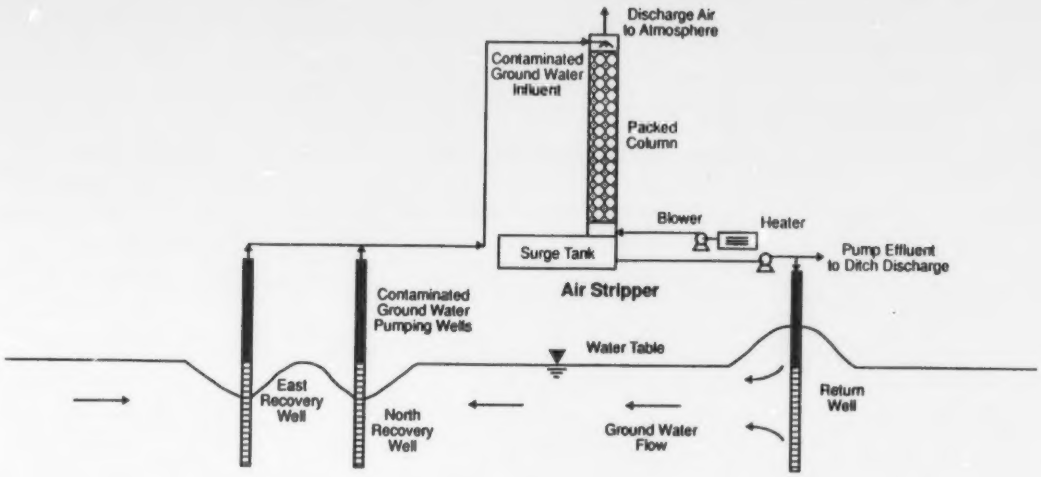

Discharge Air to Atmosphere
Contaminated Ground Water Influent
Packed Column
Blower
Heater
Surge Tank
Pump Effluent to Ditch Discharge
Air Stripper
Contaminated Ground Water Pumping Wells
Water Table
East Recovery Well
North Recovery Well
Ground Water Flow
Return Well

Figure 10
Air Stripping Installation

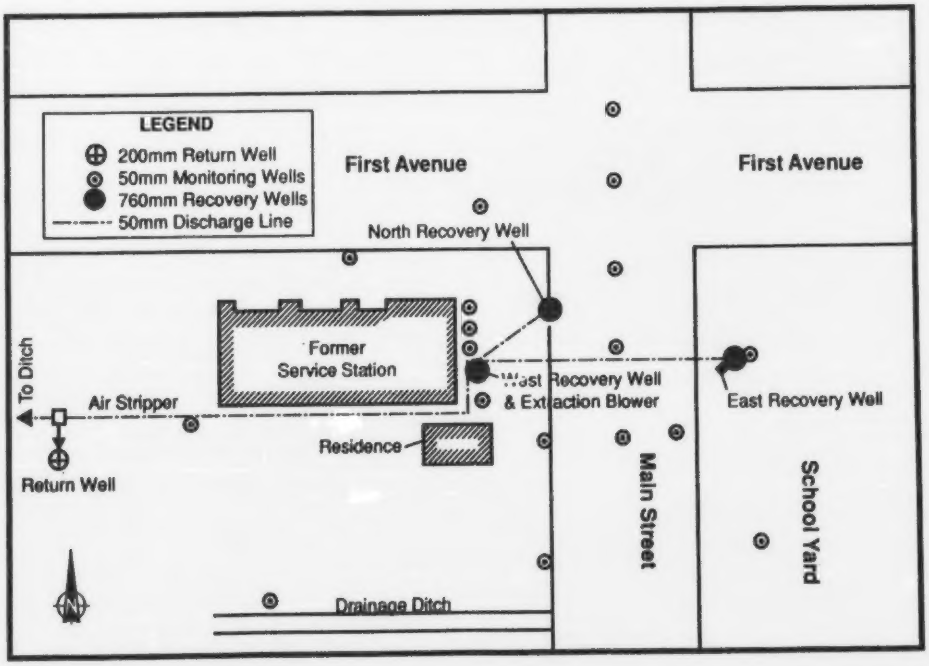


Figure 11
Site Plan Showing Remediation Equipment Installation

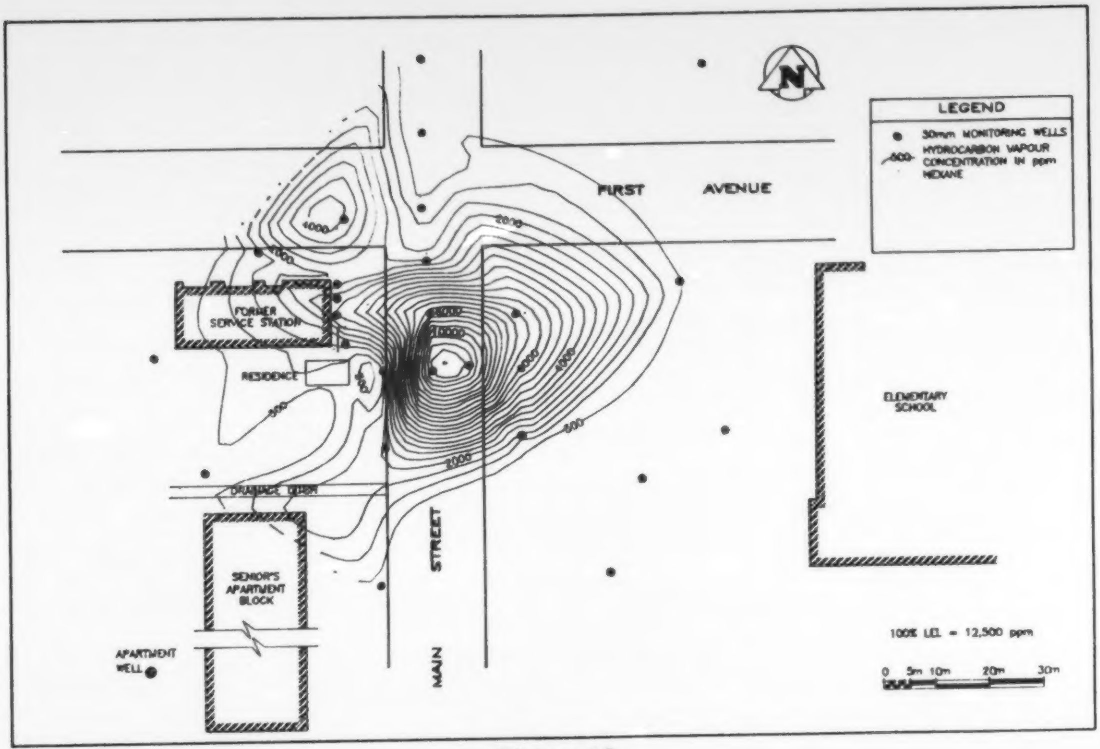


Figure 12
Hydrocarbon Vapour Concentrations, October, 1990

Sample Location	Aromatic Hydrocarbon Analysis [Compound Concentrations In Parts Per Billion (ppb)]			
	Benzene	Toluene	Ethylbenzene	Xylenes
Adjacent Residence	ND	ND	ND	ND
Senior Apartment Block	ND	ND	ND	ND
Former Service Station	ND	1.5	ND	ND
Elementary School	*0.2	1.4	ND	ND
Ditch Discharge	0.3	ND	ND	ND
Travel Blank	ND	ND	ND	ND
Field Blank	0.2	2.7	ND	1.1
Method Detection Limit	0.2	0.2	0.2	0.2
Canadian Drinking Water Quality Guidelines: Maximum Acceptable Concentration	5.0	24.0	2.4	300.0

* (ND)
ND = Not Detected
- = No Guideline

TABLE 1
Domestic Water Sample Analyses

PROPER APPLICATION OF PROGRESSING CAVITY PUMPS

David Meffen
Robbins & Myers Canada Ltd.

PROGRESSING CAVITY PUMP OVERVIEW

The progressing cavity pump was invented in 1979 by Renee Moineau, a French scientist. A progressing cavity pump is a positive displacement pump. This means that a fixed volume is displaced with each revolution. The pump is occasionally referred to as a single screw pump. The rotor, a single helix, rolls eccentrically in the stator, a double helix. This combination creates pockets or cavities which move toward the discharge end of the pump as the rotor turns. This is how this pump is commonly referred to as a progressing cavity pump.

The progressing cavity pumping principle offers more features and capabilities than any other type of positive displacement pump.

PUMP CAPABILITIES & FEATURES

- High Pressure to 1,500 PSI
- High Capacity to 2760 USGPM
- Moves Viscous Fluids to 1,000,000 CP
- Moves Abrasive Fluids
- Non-Pulsating Flow
- Low NPSH Requirement
- Flow Proportional to Speed
- Discharge Pressure Independent of Speed
- No Valves
- Quiet Operation
- Reversible
- Low Shear Pumping Action
- Passes Solids to 1.8 Inch
- Will Not Vapor Lock
- Suction Lifts to 28 Feet
- Pumps Corrosive Chemicals -- Full PH Range
- Fluid Temperatures to 425°F
- Self Priming
- High Solids Concentration
- Flow Slightly Affected By Pressure

Advantages Over Centrifugal Pumps

- Capable of much higher viscosities
- Better suited for high solids concentration
- Higher volumetric efficiency, especially with high viscosity fluids
- Better suited for shear sensitive fluids
- Better suction capabilities
- Flow is only slightly affected by pressure changes
- Capacity proportional to speed
- Pumps against pressure independent of speed
- Flow can be reversed
- Can run several pumps in parallel without difficulties on loss of performance

Advantages Over Lobe Pumps

- Better suited for handling abrasive fluid
- Can operate against higher pressure
- Better suction capabilities
- Only one shaft seal which operates on the low pressure, suction side of pump
- No timing gears to align
- Longer life in abrasive applications
- Less costly to repair. No housings, cover plates, bearings, oil seals or gears to replace or rebuild.

Advantages Over Piston Pumps

- Non-pulsating flow
- No valves
- Daily maintenance/lubrication not required
- Better suited for high solids concentration
- Better suited for shear sensitive fluids
- Better suction capabilities
- Smaller space and weight requirement
- Lower cost
- Flow can be reversed
- Quieter operation

Advantages Over Gear Pumps

- Longer life in abrasive applications
- Better suited for shear sensitive fluids
- Better suited for high solids concentration
- Higher volumetric efficiency especially with low viscosity fluid
- Quieter operation
- Bearing isolated from fluid being pumped

COMMON APPLICATIONS FOR PROGRESSING CAVITY PUMPS IN THE WASTEWATER TREATMENT INDUSTRY

PROGRESSING CAVITY PUMPS ARE COMMONLY USED IN THE FOLLOWING WASTEWATER TREATMENT APPLICATIONS.

Sludge

- primary
- secondary
- thickened
- waste-activated
- digested
- DAF
- mixed liquor
- feeding dewatering equipment
  - belt presses
  - centrifuges
  - vacuum filters
  - plate & frame filterpresses
  - etc.
- Feeding incinerators
- feeding heat exchangers

Chemicals-metering/dosing & transfer

- polymer
- lime slurry
- ferric chloride
- alum
- others

Scum & Grease

Raw Sewage

- high head lift stations
- sampling

Dewatered Sludge (Cake)

- from belt presses
- from centrifuges
- from vacuum filters
- from plate & frame filterpresses
- etc.

Pump/grinder combinations

SELECTION AND SYSTEM GUIDELINES

OVERVIEW

The selection of a P.C. pump can be an involved process. It should be recognized that most applications can be handled by more than one pump. The best selection is often determined by considering price, desired life, and operating costs. Published performance data is the average performance for a given pump size. Therefore when using performance curves, be aware that actual performance may vary slightly from that indicated by the published data.

APPLICATION DATA

The most important step is collecting all the available data for the application. some of the application conditions will affect pump size. Others will affect the power requirements. Some affect both size and power. Conditions affecting power include the flow rate, pressure, fluid viscosity, the amount and size of solids, whether the fluid is a slurry of viscous fluid, temperature and abrasion. Therefore, obtaining as much data as possible is essential to proper pump selection.

MATERIALS OF CONSTRUCTION

Materials of construction affect the pump selection due to different physical characteristics of the available materials. The stator material used can affect abrasion resistance, chemical capability, and pressure capability. The drive train materials of construction affect the torque limitation of the drive end thus affecting the size of the pump. The best source of information concerning materials of construction is often the user. Most are familiar with the fluid to be pumped and are aware of the most compatible materials. However if any doubt or question arises, a fluid sample should be tested in a laboratory for best selection.

ADJUSTING ELEMENT SIZE FOR SOLIDS

The 'maximum' particle size expected to pass through the pump must be compared to the maximum size the pumping element can accommodate and the sizing of the pump adjusted accordingly.

ADJUSTING ELEMENT SIZE FOR ABRASION AND VISCOSITY

Pump size is affected by abrasion and viscosity. Abrasive fluids cause progressing cavity pumps to wear. The best way to minimize the wear and extend the life of a progressing cavity pump is to select a larger pump that will operate at slower speeds. Viscous fluids also impact the size and speed of the pump, viscosity is the property of the fluid which causes it to resist flowing into the pump. The more viscous a fluid, the slower the pump will have to run to allow time for the fluid to flow into the pump. Even at reduced speeds the pump may not develop 100% volumetric efficiency and this must be accounted for in the selection process.

DETERMINING THE NUMBER OF STAGES

The term stages in a P.C. pump refers to the number of pitch lengths on the rotor A stage is 3 pitch lengths and each stage is rated as being capable of overcoming a head pressure of 87 PSI (6 BAR) in a clear, water like fluid but is derated from this when pumping abrasive fluids or the stator has of a lower durometer than the normal 70 durometer eg. natural rubber is 55 durometer and the pressure per stage is lowered to only 60 PSI. Determine the number of stages by dividing the required maximum differential pressure by the allowable amount for one stage for the abrasive condition of the fluid and stator material. If the result is fractional round up to the next stage. The above selection step is very important as along with pump speed the amount of slip caused by pumping against too high a pressure for the number of stages and the abrasive nature of the fluid is the most common cause of early pump failure.

DETERMINING THE PUMP SPEED

As mentioned above, speed of the pump is important in applications where abrasive of viscous fluid is being pumped. The actual speed is determined by the capacity of fluid to be pumped using the element size selected then comparing this to the maximum recommended speed for the abrasiveness or viscosity of the fluid being pumped. If the speed exceeds the recommendations then use a larger size pump to bring the speed down to within the limits.

DETERMINING HORSEPOWER REQUIRED

Establish the horsepower required for pumping water at 68°F, from the standard published curves. Then using adders, as given by the pump manufacturer, adjust the required horsepower for viscosity, solids content, solids size, and fluid temperature. After completing this calculation, determine whether the drive components of the pump can deliver the power to the pumping elements satisfactorily. If not, use the next size larger drive end in some cases it may be necessary to jump 2 sizes. It is also recommended that a larger drive end be selected if frequent cycling of the pump is expected.

SELECTION COMPARISON MEDIUM VS. HEAVY ABRASION

Sludge

The engineer must first consider the abrasive characteristics of the sludge. For purposes of selecting progressing cavity pumps, most municipal sludges are considered to be of medium abrasion.

Some municipal sludges are considered to be of heavy abrasion. The following are common causes of sludge of heavy abrasion:

--combined storm & sanitary sewers
--ineffective or no grit system
--rocks
--snails
--trash (tramp metal, bottle caps, sticks, wire, glass, etc.)

Progressing cavity pump selection is different for sludge of medium abrasion than it is for sludge of heavy abrasion.

Medium Abrasion

casting material: cast iron
rotor material: tool steel, chromaplated
stator material: nitrile/Buna N rubber, 70 durometer
shaft material: tool steel, chromaplated on stuffing box area; consider shaft sleeve
sealing: packing (grease lubricated) or mechanical seal; consider water flush system
max. pressure per pump stage: 35 PSI
max. pump RPM: 300

Heavy Abrasion

casting material: cast iron
rotor material: tool steel, K-37 ceramic coating, oversize
stator material: nitrile/Buna N rubber, 55 durometer
shaft material: tool steel with shaft sleeve
sealing: packing with water flush system
max. pressure per pump stage: 15 PSI
max. pump RPM: 200

General Specifications for Sludge

- specify crown gear-type universal joints
- specify inspection ports/clean-out ports/handholes on either side of the suction casting
- specify hollow shaft
- if sludge has more than 10% solids, specify open throat, hopper-type suction port
- specify pressure relief system (pressure sensor/switch, rupture disc, pressure relief valve) on discharge side of pump
- if there is a chance that the pump could accidentally run dry, specify a flow detection device on discharge side of pump

Special Conditions

high rag/hair/stringy material content

- consider sludge grinding
- if there is no sludge grinding, specify protective cover on universal joint-shaft area

NOTE: Inspection ports/clean-out ports/handholes and hollow shaft are strongly recommended when sludge has high content of rags, hair, or stringy material.

high rock or trash (tramp metal, bottle caps, sticks, wire, glass, etc.) content

- consider sludge grinding
- specify a trap (inverted tee) in the suction line

suction lifting/long suction lines/negative suction conditions

- if packing is used as the sealing device and the discharge pressure is less than 50 PSI, specify the pump to operate in reverse rotation (this puts the stuffing box on the pressure side of the pump, eliminating the possibility of pulling air through the packing)

  or

- utilize a mechanical seal as the sealing device

  or

- specify a water seal system for the packing (this is not as fail-safe as reverse rotation)

Required Space

If space is limited then consider the following:

- specify piggyback/overhead mounting of driver to minimize width or length
- specify hollow shaft to minimize length
- consider wall or ceiling mounting of pump unit
- consider mounting pump unit above tank and suction lifting the sludge
- do not consider "short version" progressing cavity pumps, as drive train problems will be experienced

Corrosion

It is rare for a municipal wastewater sludge to be more than slightly corrosive. Except in these rare cases, stainless steel is inferior to tool steel, because it is softer and will wear more quickly due to the abrasive characteristics of sludge. Even though it is more expensive, stainless steel wearing parts will not last as long as tool steel ones.

Progressing cavity pumps are commonly supplied in stainless steel or other materials, when corrosion so dictates.

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